ENGINEERING
RESEARCH

ENGINEERING
RESEARCH
ENGINEERING
RESEARCH
ENGINEERING
RESEARCH
ENGINEERING
RESEARCH

IOWA INTERSTATE REST AREA STABILIZATION PONDS
Part I: Pond Design
Part II: Feasibility of Wind-Powered Aeration

John L. Cleasby
Professor of Civil Engineering

Herman Dharmarajah
Research Graduate Assistant

September 1979

Prepared for
Iowa Department of Transportation
Highway Division
Contract Agreement HR-207

ISU-ERI-AMES 80028

DEPARTMENT OF CIVIL ENGINEERING
ENGINEERING RESEARCH INSTITUTE
IOWA STATE UNIVERSITY, AMES

TABLE OF CONTENTS

Preface iii

Acknowledgment v

PART I. THE DESIGN OF REST AREA STABILIZATION PONDS

1. BOD Strength of Rest Area Sewage 3

2. BOD Load Permissible 5

3. Current and Projected BOD Loads 7

   3.1. Present BOD Loads 7

   3.2. Projected BOD Loads 11

4. Hydraulic Loads and Retention 15

   4.1. Hydraulic Loads 15

   4.2. Retention Adequacy 17

   4.3. An Operational Strategy for Maximum Retention 20

5. Conclusions and Recommendations 25

6. References 29

7. Appendices 31

   7.1. Appendix A 33

   7.2. Appendix B 34

PART II. THE FEASIBILITY OF WIND-POWERED AERATION FOR REST AREA STABILIZATION PONDS

1. Introduction 39

   1.1. Power from the Wind. 39

   1.2. General Characteristics of the Wind 40

2. Theoretical Aspects of Wind Power 45

2.1. Energy Content of Wind 45

2.2. Types of Wind Driven Machines 49

2.3. The Power Coefficient 55

2.4. The Torque Coefficient 59

2.5. Relative Advantages of Different Machines 60

2.6. Power Calculations 64

2.7. Potential Power of S-Type Rotors 71

3. Economic Design of Wind-Power Systems 73

4. Application of Wind Power to Wastewater Treatment 77

4.1. Notes on Iowa Wind Pattern 77

4.2. Previous Use of Wind Power for Aeration of Wastewaters 77

4.3. Use of Wind Power for Aerating Rest Area Stabilization Ponds in Iowa 82

5. Typical Capital and Operating Costs 87

6. Conclusions 91

7. References 93

8. Appendix 95

PART III. SUPPLEMENT

1. Travel Trailer Disposal Load to Interstate Rest Area Lagoons 106

PREFACE

This report is presented in two parts. Part I takes a new look at the design of rest area stabilization ponds after nearly 10 years' experience with some of the existing ponds and in the light of new design standards issued by Iowa DEQ. The Iowa DOT is embarking on improvements to the ponds at some of the rest areas. These improvements may include installation of drainage tile around the ponds to lower the water table below the pond bottom, sealing of the ponds with bentonite clay to reduce the infiltration to limits recently established by Iowa DEQ, and the enlargement of the ponds or installation of aeration equipment to increase the pond capacity. As the Iowa DOT embarks on this improvement program, it behooves them to make only the improvements that are absolutely necessary to achieve wastewater treatment goals. These ponds are subject to an extremely seasonal load and thus the ordinary standards used for pond design are not appropriate. Thus, Part I of the report presents a rationale for design and operation of the ponds which is deemed appropriate for their unique seasonally loaded character.

Part II of the report looks at the feasibility of using wind power for the aeration of the ponds, if and when aeration is deemed necessary.

ACKNOWLEDGMENT

This research was supported in part by the Engineering Research Institute of Iowa State University with funds from the Iowa Department of Transportation, Contract HR-207. The participation of the staff of Iowa Department of Transportation and Iowa Highway Research Board is also acknowledged.

PART I

THE DESIGN OF REST AREA STABILIZATION PONDS

1. BOD_5 STRENGTH OF REST AREA SEWAGE

The design of the rest area stabilization ponds is controlled by either the BOD_5 load or the hydraulic load. The new Iowa design standards for wastewater treatment ponds [5] permit only controlled discharge ponds or aerated facultative ponds. Two-cell controlled discharge ponds are permitted for small installations (< 1 acre total surface area) with a maximum organic load of 20 lb BOD_5/acre/day on the primary cell. Thus, one aspect of design that must be considered is the BOD_5 load on the first cell which in turn is dependent upon the BOD_5 strength of the wastewater.

A review of available literature [3,4,6,9,10] containing information on the BOD_5 of composite samples of rest area wastewater revealed a wide range of reported values from 78 to 210 mg/l with an average of 154 mg/l (standard deviation = 34.6, n = 17), and a median value of 161 mg/l. These data reinforce the conclusion of the 1977 Corps of Engineers report [4] which suggested that the raw wastewater will range from 125 to 175 mg/l BOD_5.

The data include two reports of sampling at Iowa rest areas. Parker [10] reported BOD values of 130 and 110 mg/l which were originally presented by J. T. Pfeffer (Illinois Highway Report IHR-701, 31 March 1973). Hughes et al. [3,4] reported Iowa values from 59 to 561 mg/l with a mean of 210 (s = 137) for data originally presented by R. Zaltzman in April 1975. However, a phone call to Mr. Zaltzman at West Virginia University revealed some difficulty in the Iowa sampling created by the temporary retention and periodic discharge of solids from the manhole located ahead of the sampling site. Such a situation would contribute to erratic and somewhat undependable results. Forty-two grab samples at a Mississippi rest area had BOD_5 ranging from 12 to 432 with a median of 96 mg/l [4].

On the basis of the foregoing information, a BOD_5 value of 170 mg/l is suggested by the authors for evaluation of BOD load on the Iowa interstate rest area ponds. This is close to the high end of the range suggested by the Corps of Engineers report [3], a little above the average and median value reported for composite samples, and well above the Pfeffer data for Iowa rest areas. So, 170 mg/l BOD_5 should be adequately conservative.

One point of substantial uncertainty is the volume and BOD strength of the wastes contributed at camper dump stations. Only one quantitative piece of information was found about the strength of such wastes. Hughes [4] presented data for grab samples at a Mississippi rest area. One high BOD value of 1965 mg/l was identified as occurring during a "trailer dump." It would be dangerous to use a single value to predict the BOD load from this source, but it does indicate that the potential load from such sources is significant and should be evaluated.

2. BOD LOAD PERMISSIBLE

As stated previously, the new Iowa DEQ design standards allow a load of 20 lb BOD_5/acre/day on the primary cell of a small two-cell controlled discharge pond [5]. This criterion was evolved for municipal systems where load is fairly uniform through the 12 months of the year. The rest area ponds receive a highly seasonal load, the major load coming during the summer months of June, July, and August.

It is during these months that ponds are most able to cope with high BOD loads. This is shown by the common design criteria of 50 lb BOD_5/acre/day which was used until recent years for municipal ponds in the southern United States [1]. It should also be remembered that this criterion was based on the load to the total pond area rather than only to the primary cell. Therefore, if the common two-cell lagoon system in use at that time was operated in series, the first cell could receive loads up to 100 lb BOD_5/acre/day.

The ability of ponds to handle high loads in the summer is also evidenced by more rational design approaches based upon amount of solar radiation expected [7,8] temperature and other factors [12]. For example, Neel presented an exhaustive study of five small one-acre experimental ponds at loadings from 20 to 100 lb BOD/acre/day and concluded that if ice cover need not be considered it would appear that a minimum monthly solar radiation level averaging around 150 to 160 langleys/day (based on total spectrum) would furnish enough light to maintain oxygen in ponds loaded up to 100 lb BOD_5/acre/day [8].

During the summer months, total solar radiation in Iowa on days with no sunshine ranges from 140 to 173 langleys/day [7] and with full sunshine from 250 to 298 langleys/day. Therefore, loads of 100 BOD_5/acre/day should be possible.

As a final piece of evidence, Gloyna gives a loading range of 136-320 lb BOD/acre/day for tropical climates with uniformly distributed sunshine and temperature and no seasonal cloud cover, i.e., no cloud cover for extended periods [2, p. 64].

On the basis of the foregoing information, it would be reasonable to accept BOD loads of up to 100 lb BOD_5/acre/day on the primary cell of a two-cell pond for the peak three months of summer, provided that the average annual load remained near the normal range of 20 lb BOD_5/acre/day. This low annual load is suggested to prevent excessive bottom deposits from developing during the winter months which would then contribute (feed back) additional load during the spring and summer months.

3. CURRENT AND PROJECTED BOD LOADS

3.1. Present BOD Loads

The current BOD loads to the Iowa DOT rest area lagoons have been calculated using the following approach:

1. The actual water pumpage records were used to predict wastewater volume (1977 and 1978 data are summarized in Table 1). It was assumed that 82 percent of water pumped becomes wastewater to the pond based on Zaltzman's observations summarized by Hughes et al. [4].

2. A raw BOD_5 of 170 mg/1 was assumed based on the data analyzed in Section 1.

3. A primary pond area of 1/4 acre at a water depth of 5 ft was used on each side of the Interstate (or in a few cases a 1/2 acre primary pond served both sides of the Interstate).

4. A 50-50 split of load between the two sides of the highway was used except for Davenport and Ames which exhibit an unbalanced split (see Appendix A).

The calculation approach for average annual load is given at the bottom of Table 2. To obtain the load during the three months of the summer (June, July and August) an analysis of the water pumpage records was made to determine what fraction of the demand occurs during those three months (see Appendix B). The fractions ranged from 30 to 57 percent with a median of 46% in 1976 and 47% in 1977. Therefore, a 50 percent demand during those months was used as a conservative value in developing Table 2.

The data of Table 2 indicate that on the basis of both the annual average load (all are under 20 lb/acre/day) and the peak three-month load (all are under 40 lb/acre/day, well below the permissible 100 lb/acre/day discussed in Section 2) the primary ponds are not heavily

Table 1. Water pumpage at Iowa interstate rest areas.

Rest Area	1977 AADT*	1977 Water Pumpage Mil. Gallons	gal./day / AADT	1978 AADT	1978 Pumpage Mil. Gallons	gal./day / AADT
Adair	11300	2.438	0.59	11200	2.188	0.54
Cedar	15600	1.656	0.29	16600	1.718	0.28
Victor	14700	1.957	0.36	14800	2.054	0.38
Grinnell	14900	1.493	0.27	15100	1.483	0.27
Tiffin	15200	1.637	0.30	15700	1.710	0.30
Mitchellville	18300	1.851	0.28	17900	1.874	0.29
Davenport	11500	1.802	0.43	12500	1.871	0.41
Sgt. Bluff	8280	0.886	0.29	9900	0.885	0.23
Ankeny	12700	1.537	0.33	15300	1.402	0.25
Ames	9980	2.190	0.60	10500	2.235	0.58
Dallas Co.	15200	1.721	0.31	15500	1.861	0.33
Loveland	2900	1.105	1.04	3770	1.103	0.80
Mo. Valley	5990	1.236	0.57	6240	0.990	0.43
Onawa	5500	0.711	0.35	5700	0.766	0.37
Underwood	9590	2.018	0.58	9390	2.749	0.80
Osceola	5390	0.997	0.51	5800	1.205	0.57
Decatur	4370	1.060	0.66	5260	1.034	0.54
Pacific Jct.	8910	1.562	0.48	9530	1.336	0.38
Clear Lake	5990	2.462	1.13	7100	2.603	1.00
Linn Co.	11800	0.893	0.21	12500	0.995	0.20
		Average	0.48			0.45
		Std. Deviation	0.25			0.22
		Median	0.39			0.38

*AADT = Average Annual Daily Traffic

Table 2. Calculated BOD_5 loads to primary cell of Iowa interstate rest areas (excluding load from camper dump stations).

Rest Area	lb BOD_5/acre/day Avg. Annual* 1977	1978	June July Aug.** 1977	1978	Camper Dump Station
Adair	15.5	14.0	31.0	28.0	Proposed
Cedar	10.5	11.0	21.0	22.0	Proposed
Victor	12.4	13.0	24.8	26.0	Proposed
Grinnell	9.5	9.4	19.0	18.8	Proposed
Tiffin	10.4	10.9	20.8	21.8	Yes
Mitchellville	11.7	12.0	23.4	24.0	Proposed
Davenport (N. Side)	15.6	16.0	31.2	32.0	Proposed
(S. Side)	7.3	7.6	14.6	15.2	Proposed
Sgt. Bluff	5.6	5.6	11.2	11.2	Proposed
Ankeny	9.7	9.0	19.4	18.0	Proposed
Ames (W. Side)	17.5	17.9	35.0	35.8	Yes
(E. Side)	10.3	10.5	20.6	21.0	Yes
Dallas	10.9	11.8	21.8	23.6	Yes
Loveland	7.0	7.0	14.0	14.0	Proposed (S. Side only)
Mo. Valley	7.8	6.3	15.6	12.6	Yes
Onawa	4.5	4.9	9.0	9.8	Yes
Underwood	12.8	17.5	25.6	35.0	Yes
Osceola	6.3	7.6	12.6	15.2	Yes
Decatur	6.7	6.6	13.4	13.2	Yes
Pacific Jct.	9.9	8.5	19.8	17.0	Yes
Clear Lake	15.6	16.6	31.2	33.2	Yes
Linn Co.	5.7	6.0	11.4	12.0	Yes

$$* \left(\frac{\text{Total annual pumpage to both sides from Table 1, MG/yr}}{365\ \text{day/yr}}\right)\left(\begin{matrix}\text{Fraction}\\ \text{split}\\ \text{between}\\ \text{slides}\end{matrix}\right)\left(0.82\ \frac{\text{waste}}{\text{water}}\right)\left(170\ \frac{\text{mg}}{\text{l}}\right)\left(8.33\ \frac{\text{lb/MG}}{\text{mg/l}}\right)\left(\frac{1}{0.25\ \text{acres}}\right)$$

** Same approach except assume 1/2 load arrives in 3 summer months.

loaded at the present time. However, the loads in Table 2 do not include any contribution from camper dump stations because there was insufficient information to calculate such loads with any reliability. Nevertheless, some attempt must be made to estimate the camper dump station load.

Rest area supervisors estimate use of camper dump stations at 15 to 25 dumps per day, with a typical tank size of 30 gallons (H. Dolling, Iowa DOT, personal conservation). If an average summer usage of 20 dumps per day is assumed per dump station, with an average volume of 15 gallons and a strength of 2000 mg/l BOD_5 (based on one sample reported by Hughes [4]), the BOD load would be about 5.0 lb/day or 20 lb BOD_5/acre/day for a quarter acre pond. This load would be in addition to the summer load shown in Table 2. It is therefore evident that the load from the camper dump stations could be substantial and more data should be collected on frequency of use, size of tanks and strength of wastes if rational load projections are to be made for future designs.

Considering this crude estimate of camper dump load, some rest areas are already receiving summer loads of 40 to 60 lb BOD/acre/day (Tiffin, Ames, Dallas, Underwood and Clear Lake), yet these areas are functioning without complaints of odor nuisance as reported in questionnaire responses dated May 15, 1979, collected by Wayne Sunday from the rest area supervisors. The only rest areas that reported substantial and persistent odor problems were Grinnell and Adair (east bound). Both of these rest areas are served by water supplies that are unusually high in sulfate (SO_4) content (3000 mg/l Grinnell north side, 1500 mg/l at Adair). Since the loads on these lagoons are not higher than several other rest areas without problems, one can conclude that the odors experienced thus far are more related to the high SO_4 water than to the prevailing BOD load level. The sulfates are reduced to sulfide in the anaerobic bottom layers of the pond, producing hydrogen sulfide (H_2S) gas with a characteristic rotten egg odor.

The Grinnell rest area provides an interesting case. The north side area served by the 3000 mg/1 SO_4 water has always generated more odor than the south side area which is served by a water with 770 mg/1 SO_4. Both areas have low BOD and hydraulic loads. One other difference may contribute to the degree of the odor problem. The north area is in a low area sheltered by trees and by a steep hillside from the prevailing southwest winds of the summer. Odors are most noticeable on hot muggy days. The south side ponds are in the open with a free sweep of the southwest winds over the ponds.

Thus, the experience thus far would support the acceptability of high summer season loads except in special areas complicated by high sulfate water or special topographic situations.

3.2. Projected BOD Loads

The projected loads in the future will increase roughly in proportion to the average annual daily traffic (AADT) and the fraction of traffic using the rest areas. Using the projected AADT figures of Cedar County and Davenport as a guide, we can expect the AADT to increase about 80 to 90 percent by 1999. If the projected traffic growth is similar in other rest areas, the 1999 primary pond BOD loads (excluding camper dump load) at some rest areas will reach 60 to 70 lb/acre/day in the summer three months, and several will exceed 20 lb/acre/day on an average annual basis (to obtain these loads, multiply the values in Table 2 by the ratio of projected AADT/present AADT).

The load from camper dump stations must then be added to the above loads. If the summer camper dumping load is about 20 lb BOD/acre/day as crudely estimated before, some summer loads to the primary cell will approach 80 to 90 lb BOD/acre/day. Assuming that the camper dumping load occurs almost entirely in the summer, the 20 lb/acre/day summer load would add about 5 lb BOD/acre/day to the annual average loads.

Since some rest areas may approach or exceed the selected permissible loads of 100 lb/acre/day in the summer and 20 lb/acre/day on an annual average basis, a suitable strategy for these potential overloads should be formulated. Several possibilities are evident:

1. In view of the uncertainty of projected loads created partly by the camper dumping load question and partly by the model used to generate Table 2, one strategy would be a "wait and see" plan. Essentially, this means one would <u>not</u> embark on enlargements or aeration of the rest area ponds based on these projected loads. Rather, one would take an observational approach and, as the loads increase in the future, record any observation of odor nuisance and take remedial action only when the frequency or intensity of such nuisance indicates that the load is excessive.

This "wait and see" approach is also encouraged by the uncertainty about traffic and recreational vehicle usage in the future created by higher gasoline prices of recent and coming years.

The suitability of the 20 lb BOD/acre/day annual average load to the primary cell could also be questioned because such a large share of the load comes in the warmer months. This summer load is largely treated by aerobic and anaerobic mechanisms before the onset of the winter months when loads will be well below the 20 lb limit.

2. A second strategy would be to enlarge the primary ponds when the loads approach or exceed the 20 lb BOD/acre/day annual average load limit. However, this alternative should <u>not</u> be instituted without attempting to gather better data on the contribution of camper dump stations to the total load.

3. The use of aeration equipment in the ponds does not seem justified under the BOD criteria outlined above (100 lb summer and 20 lb annual average load). However, for those cases where odor nuisances become serious because of high SO_4 water supplies or topographic conditions, installation of aerators for operation only during the nuisance periods may be necessary.

Low flush toilets are proposed in the improvements of the Cedar and Davenport rest areas to increase the retention period in the ponds. It is difficult to predict the full impact of this proposal on the pond performance. Decreasing the amount of water does not increase the BOD load, but it does increase the BOD strength. Since the BOD load is not increased, the oxygen requirement in the pond is not increased. First order kinetic models for BOD reduction would predict that a reduction in flush water volume would improve the effluent quality because of increased detention time, in spite of the fact that the BOD concentration is increased. For example, Thirumurthi [12] proposed several kinetic models for design of stabilization ponds. His approximate equation is a first-order kinetic model:

$$\frac{C_e}{C_i} = e^{-kt} \tag{1}$$

where:

C_e = effluent BOD concentration

C_i = influent BOD concentration

k = first order removal coefficient (days^{-1})

t = mean detention time (days)

If the flush water is reduced by half with a resultant doubling of both C_i and t, and if any typical value of k is assumed, it can easily be demonstrated that C_e will always be better with the doubled waste strength and doubled detention time.

So it appears that the proposal to reduce the flush water volume will enhance the performance of the lagoons. However, if the "wait and see" strategy is adopted, one could add aeration equipment at a later date if unforeseen nuisance conditions develop.

4. HYDRAULIC LOAD AND RETENTION

4.1. Hydraulic Loads

The design of Controlled Discharge Ponds involves consideration of the BOD load to the primary cell as well as the ability of the pond to retain the flow between the times of controlled discharge [5]. These standards require 180 days of hydraulic storage above the 2 ft depth during the wettest 180 consecutive days of the year. Normally, the controlled discharge is allowed in the spring and fall seasons when receiving waters will provide more dilution and when pond algae populations are lower. According to the DEQ Design Manual, chap. 18C 5.3.1. [5], the 180-day retention is calculated from the 2 ft minimum water level to the normal high water level in the entire pond system (the system is defined to include both cells).

The new design standards also limit infiltration to a maximum of 1/16 inch per day for the pond area when the pond is at a water depth of 6 ft. However, Iowa DEQ practice does not include this water loss in determining the required pond volume.

The permissible hydraulic loads to the Iowa Interstate rest area ponds can be generalized in the following manner:

1. At each rest area, there are 2 cells of 1/4 acre on each side of the highway (or in some cases, 2 cells of 1/2 acre serving both sides).
2. The storage volume between the 2 ft and 5 ft level for a 1/4-acre pond is typically about 190,000 gallons per 1/4-acre cell depending on the shape of the pond, or about 760,000

gallons per pair of rest areas. This presumes that both the primary and secondary cells will be drawn down in the spring to enter the peak season with maximum storage capacity available. (An operational strategy to do this will be discussed later).

3. During the 6-month wet season, the total rainfall on the ponds is approximately equal to or slightly less than the evaporative loss [11] so the contribution from those two sources to the water balance can be ignored.
4. The allowable infiltration at 1/16 inch per day on 1/4 acre is equal to 424 gal./cell/day or 1696 gal./day/rest area, but this loss will be ignored in accordance with Iowa DEQ practice.

Thus, if the goal of 180-day hydraulic storage is to be achieved, the total wastewater flow per pair of rest areas could not exceed 760,000 gallons in 180 days.

If wastewater is again assumed to represent about 82 percent of water production, the 180-day water production should not exceed 760,000/0.82 = 927,000 gallons total for the two sides of the rest area or 463,000 gallons for one side of the rest area.

If the 1/16 inch per day infiltration is included in the water balance, the resulting permissible water productions would be 1,299,000 gallons for the two sides of the rest area, or 644,000 gallons for one side of the rest area.

To judge the adequacy of the present ponds to meet the 180-day storage criterion, the water production for 180 days must first be determined.

It must again be emphasized that the hydraulic load is quite seasonal with about 46 percent of the water demand occurring during the three peak months of June, July and August (Appendix B); and about 72 percent occurring during the six peak months of May through October (Appendix B). Applying these median percentages to the 1978 water demand data of Table 1 yields the data presented in Table 3.

4.2. Retention Adequacy

If the 180-day water demands of 1978 in Table 3 are compared with the acceptable water production criteria estimated previously, it is evident that fourteen of the rest areas are already violating the 180-day storage criterion which ignores the contribution of infiltration to the water balance. If the contribution of infiltration is included in the water balance, only eight rest areas are potentially in violation. However, only three rest areas have found it necessary to discharge more frequently than twice a year (Adair, Mitchellville, and Victor). Thus, it appears several may actually exceed the 1/16 inch per day allowable infiltration. Other factors are also involved. For example, in some hot dry years, evaporation can exceed precipitation by about 20 inches during April through October, the equivalent of about 1/8 inch per day on the pond surface. At the other extreme, in some cool wet years, precipitation can exceed evaporation by about 12 inches. This situation could be handled in the normal freeboard provided, thus avoiding discharge during the summer.

Table 3. 1978 water demand (million gallons).

Rest Area	Total Annual	Maximum 3 Months	Maximum 6 Months
Adair	2.188	1.006	1.575**
Cedar	1.718	0.790	1.237*
Victor	2.054	0.945	1.479**
Grinnell	1.483	0.682	1.068*
Tiffin	1.710	0.787	1.231*
Mitchellville	1.874	0.862	1.349**
Davenport			
S. Side	0.622	0.286	0.448
N. Side	1.249	0.574	0.899**
Sgt. Bluff	0.885	0.407	0.637
Ankeny	1.402	0.645	1.009*
Ames			
W. Side	1.406	0.647	1.012**
E. Side	0.829	0.381	0.597*
Dallas Co.	1.861	0.856	1.339**
Loveland	1.103	0.507	0.794
Mo. Valley	0.990	0.455	0.713
Onawa	0.766	0.352	0.551
Underwood	2.749	1.264	1.979**
Osceola	1.205	0.554	0.868
Decatur	1.034	0.476	0.745
Pacific Jct.	1.336	0.615	0.962*
Clear Lake	2.603	1.197	1.874**
Linn Co.	0.995	0.458	0.716

* Exceeds permissible 180-day water demand, neglecting infiltration.

**Exceeds permissible 180-day water demand, including infiltration contribution.

It is apparent from Table 3 that many rest areas will not be overloaded on a hydraulic storage basis for years to come. For those that are already overloaded or may soon be overloaded, three alternatives exist.

The first alternative is to enlarge the pond volume to increase the hydraulic storage. Various options are available of course. The new standards allow 6 ft maximum water depth in the primary cell and 8 ft maximum water depth in the secondary cell [5, chap. 18. C.5.4]. There is a distinct advantage in increasing the depth of the secondary cell if the ponds are to be operated as 180-day controlled discharge ponds. The deeper secondary cell allows a greater volume of water to be discharged before the depths of the two cells are equalized. Thus in the fall and spring drawdown described in the next section, it would be sufficient to make only two discharges in the fall and two in the spring to reduce the stored volume adequately to retain the inflow for the next six months.

Therefore, if topographic conditions and the outfall sewer hydraulics are favorable, lowering the bottom of the secondary pond is strongly recommended as part of the upgrading of any of the rest area pond systems. If this option is undertaken, then the deeper pond should always be operated as the secondary pond (i.e., the series sequence should never be reversed).

The second alternative to extend the retention period to 180 days is to reduce water use and wastewater production by means of low flush toilets. This procedure has already been initiated in the designs for improvements to the Scott (Davenport) and Cedar County rest areas now in the construction stage. A combination of low flush toilets and pond deepening may be desirable at some rest areas where topographic conditions and outfall sewer hydraulics are favorable.

The third alternative is to install aeration equipment in the first stage thus converting the ponds to "flow-through aerated facultative pond" systems which normally would have less stringent retention requirements. However, if the requirements formulated at the meeting with Iowa DEQ of July 19, 1979 (C. Bartel memo dated July 20, 1979) are enforced, there is no particular advantage from the standpoint of retention to using aeration equipment. This is because 90-day retention in the secondary cell was stipulated in the memo, whereas the normal requirement for controlled discharge ponds is 180-days storage for the pond system [5, chap. 18C.5.3], which includes both cells. Further, the memo states that the primary aerated cell must be held at the 5 ft water level. Therefore, it would be just as easy to meet 180-days storage in two cells without aeration as it would be to meet 90-days storage in the secondary cell with aeration. So, while aeration may be useful to prevent odor nuisance, it cannot be justified to reduce storage under the stipulations of the above memo.

4.3. An Operational Strategy for Maximum Retention

It is not sufficient merely to provide 180-days storage so that controlled discharge can occur spring and fall when the secondary pond water quality is the best and when the receiving streams can provide some dilution. An optimal operating plan must be used to take maximum advantage of the storage. The following operating plan is proposed:

Summer Operation

1. Attempt to enter the summer season with low water level in both cells.
2. Direct all raw sewage to the primary cell with the interconnection valve to the secondary cell closed.
3. As the primary cell approaches the 5 ft level, open the interconnection valve and equalize the two cells, thus lowering the primary cell to about mid-depth and raising the secondary cell to about mid-depth. Close the interconnection valve.
4. Again fill the primary cell and equalize as in step 3. If the rising level of the primary makes discharge from the secondary cell appear unavoidable before the 180-day goal, make no equalizations of water level for about three weeks prior to the controlled discharge. These three weeks will allow needed time for sampling and discharge.
5. Iowa DEQ regulations for sampling 2 weeks before discharge, sampling during the discharge and the maximum rate of discharge must be followed. Hopefully, discharge during the summer will not be necessary after low flush toilets have been installed or other measures have been taken to provide 180-day storage. After discharging to the 2 ft level, open the interconnecting valve, equalize the levels, and close the valve.

Fall Drawdown

1. In the fall, as the rest area load decreases, and when the fall rains have provided some dilution water in the receiving stream, lower the secondary cell to the 2 ft level as early in the fall as possible (hopefully about late September). Iowa DEQ regulations regarding sampling and discharge rate must be followed.
2. Immediately equalize the depths and shut the interconnection valve. Then wait at least one week before sampling the secondary cell again in anticipation of the second fall discharge of the secondary cell. Each time this process is repeated, the primary cell will go lower. Three fall discharges before the end of November should bring both cells to near minimum level.

Winter Operation

1. Continue the raw sewage flow to the same primary cell. If and when the primary cell approaches the 5 ft level, open the interconnection valve and equalize the depths. Due to the low winter flows, this may not be necessary until spring. Close the interconnecting valve after equalizing the depths.
2. Repeat the equalizations as often as necessary during the winter, without discharging from the secondary cell.

Spring Drawdown

The spring drawdown is identical to the fall drawdown. Both cells should be brought to the lowest possible level by two or three discharges in April and May. If the secondary cell is deepened as discussed previously, the deeper cell should always be used as the secondary cell. Mr. Fred Evans of the DEQ recommends that the sequence not be reversed, even if the cells are identical. The rationale for this recommendation is to keep the secondary cell in as clean a condition as possible to obtain the best effluent BOD.

5. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions are offered which should form the basis of a new proposal to Iowa DEQ for the Iowa interstate rest area stabilization ponds:

1. The Iowa rest area ponds are unique because of the high summer season load. Thus, special design criteria should be adopted appropriate to this unique load situation.

2. Summer peak BOD loads of up to 100 lb BOD_5/acre/day can be treated without supplemental aeration because of the warm temperatures and abundant sunlight during the summer months.

3. Supplemental aeration should not be added on the basis of the summer loads projected in this report because there is too much uncertainty about the assumptions used in making the projections, particularly the loads associated with camper dump stations. Data on the frequency of use of these dump stations, volume of discharge and strength of waste should be collected to strengthen the confidence in the future load projections.

4. Supplemental aeration should be added only in those special cases where odor nuisances are excessive and persistant because of high sulfate water supplies, unusual topographic situations, or unexpectedly high BOD loads.

5. In view of the extremely low winter BOD loads, supplemental aeration will not be needed in the winter months. Therefore, in those few instances where supplemental aeration is deemed necessary in spring or summer seasons, it will be acceptable to use floating mechanical

aerators because they will not be subjected to ice problems. Floating aerators will probably be the method of choice because of their lower initial cost, efficiency of oxygen transfer, flexibility, and convenience of maintenance during the off season.

6. The program of providing tile drainage around the ponds and the bentonite sealing of the pond bottoms being initiated at the Cedar and Scott county rest areas should be expanded, especially to those rest areas receiving apparent inflow from the ground water which necessitates more frequent discharge than would be anticipated based on water usage records. It may also be necessary to repair the sewer feeding the pond system if excessive ground water is being contributed because of infiltration into the sewer system. For example, both of these problems may exist at the Mitchellville south side area and the Victor south side area.

7. A firm and positive management program should be initiated with regard to the discharge of secondary cell contents to the receiving stream or ditch. This plan should ensure that the maximum possible storage is utilized before discharge is permitted, and that the discharge is always from the secondary cell. The regulations of the Iowa DEQ for sampling before and during the discharge and for the rate of discharge should be followed.

8. For those rest area ponds being upgraded with bentonite seals, consideration should be given to deepening the secondary cell by 2 ft when topographic conditions and outfall sewer hydraulics are favorable for such a change. The deeper secondary cell will permit the two cells to be lowered adequately by two discharges in the fall and two in the

spring (rather than three each time). Thus the amount of sampling required will be reduced and pond operation will be simplified.

9. Those rest areas which have water use during the peak 6 months approaching or exceeding about 1.3 million gallons per rest area (total of both sides of the highway) or about 0.65 million gallons for one side of the highway, will need to be equipped with some low flush toilets to enable controlled discharge to occur only in the spring and fall seasons. Another alternative would be to increase the pond size to provide more storage capacity. The rest areas already in need of such provisions are Victor, Mitchellville, Davenport (Scott) north side, Ames west side, Dallas County, Underwood, and Clear Lake.

10. The use of low flush toilets will reduce the water volume delivered to the ponds but will not increase the BOD load to the ponds, nor will the oxygen requirements of the pond be increased. Reducing the volume of flush water with a proportionate increase in BOD concentration will enhance BOD reduction according to first-order kinetic models for pond performance. Thus, the quality of water leaving the primary pond will be improved by the provision of low flush toilets.

6. REFERENCES

1. Canter, L.W. and Englande, A.J. Jr., State's Design Criteria for Waste Stabilization Ponds. Water Pollution Control Federation 42: 1840-1847, October 1970.

2. Gloyna, E.F. Waste Stabilization Ponds WHO Monograph Series No. 60, Geneva, 1971.

3. Hughes, G.W., et al., Wastewater Treatment Systems for Safety Rest Areas. FHWA-RD-77-107, Final Report, Sept. 1977. Available from National Technical Information Service, Springfield, VA.

4. Hughes, G.W., et al., Rest Area Wastewater Treatment, An Overview Paper. FHWA-RD-77-107, Sept. 1977.

5. Iowa Department of Environmental Quality, Chemicals and Water Quality Division Design Manual. Iowa Wastewater Facilities Design Standards, Chap. 18C. Effective July 5, 1979.

6. Manos, G.P. Field and Laboratory Evaluation of Roadside Park and Wastewater Treatment Plants. Technical Report, University of Akron, FHWA-OH-77-01, March 1977.

7. Metcalf & Eddy, Inc. Wastewater Engineering. New York: McGraw Hill Book Company, 1972, p. 560.

8. Neel, J.K., et al., Experimental Lagooning of Raw Sewage at Fayette, Mo. J. Water Pollution Control Federation, 33: 603-641, June, 1961.

9. Parker, C.E. et al., Evaluation of water reuse concept for highway rest areas. Paper for presentation at 56th Trans. Res. Bd. Meeting, Jan. 1977. Virginia Highway and Transportation Research Council No. 77-R9P, Jan. 1977.

10. Parker, C.E., Water reuse at highway rest areas: evaluation stage. FHWA-VA-78-R22, U.S. Dept. of Commerce, NTIS, No. PB-278 542, Dec. 1977.

11. Shaw, R.H. Comparison of methods of estimating evaporation from small water surfaces in Iowa. Final report submitted to U.S. Weather Bureau, Memorandum of Understanding CWB. 8808, Aug. 1957.

12. Thirumurthi, D., Design criteria for waste stabilization ponds. J. Water Pollution Control Federation, 46: 2094-2106, Sept. 1974.

7. APPENDICES

7.1. Appendix A

Percent of total water demand on each side of the Interstate at each rest area.

	Rest Area		1974	'75	'76	'77	'78
1	Adair	S	67	59	53	56	51
2		N	33	41	47	44	49
3	Cedar	S	43	40	36	47	46
4		N	57	60	64	53	54
5	Victor	S	52	53	55	53	49
6		N	48	47	45	47	51
7	Grinnell	S	50	47	47	44	47
8		N	50	53	53	56	53
9	Tiffin	S	46	46	48	48	52
10		N	54	54	52	52	48
11	Mitchellville	S	47	52	53	51	52
12		N	53	48	47	49	48
13	Davenport	S	36	31	28	34	33
14		N	64	69	72	66	67
15	Sargent Bluff	W	55	55	54	49	56
16		E	45	45	46	51	44
17	Ankeny	W	60	52	51	52	50
18		E	40	48	49	48	50
19	Ames	W	58	55	61	64	63
20		E	42	45	39	36	37
21	Dallas	S	43	45	48	49	50
22		N	57	55	52	51	50
23	Loveland	S	55	52	50	54	50
24		N	45	48	50	46	50
25	Mo Valley	W	53	57	59	45	54
26		E	47	43	41	55	46
27	Onawa	W	48	47	47	58	44
28		E	52	53	53	42	56
29	Underwood	W	46	45	46	46	40
30		E	54	55	54	54	60
31	Osceola	W	57	53	52	48	55
32		E	43	47	48	52	45
34	Decatur						
35	Pacific Jct.	E	46	44	41	60	41
36		W	54	56	59	40	59
37	Clear Lake	W	56	56	57	59	60
38		E	44	44	43	41	40
48	Linn Co.	E	---	---	50	55	51
49		W			50	45	49

7.2. Appendix B

Seasonal variation in water demand. Percent of total annual water use occurring in indicated time period.

Rest Area		1976 3 Month J.J.A	1976 6 Month M-O	1977 3 Month J.J.A		1977 6 Month M-O	
Adair							
001R	S	49	74	51		77	
002R	N	50	81	52		77	
Cedar							
003R	S	47	72	47	50.7*	74	74.8*
004R	N	53	77	52		77	
Victor							
005R	S	44	69	47		74	
006R	N	51	76	49		74	
Grinnell							
007R	S	55	81	49		75	
008R	N	50	74	50		74	
Tiffin							
009R	S	48	73	47		73	
010R	N	51	75	49		74	
Mitchellville							
011R	S	49	73	48		73	
012R	N	49	73	49		73	
Davenport							
013R	S	52	74	45	50.11*	72	74.52*
014R	N	54	75	50		76	
Sgt. Bluff							
015R	W	51	72	50		72	
016R	E	46	69	46		70	
Ankeny							
017R	W	40	63	38		62	
018R	E	37	64	42		67	
Ames							
019R	W	Incomplete Year		42		68	
020R	E	39	67	42		68	
Dallas Co.							
021R	S	49	75	48		74	
022R	N	51	74	50		75	
Loveland							
023R	S	30	70	57		66	
024R	N	34	60	41		64	

Continued on next page.

*Based on 4 years, 1974-77, by Wayne Sunday, average of two sides.

Seasonal variation in water demand. Percent of total annual water use occurring in indicated time period. (continued)

Rest Area		1976 3 Month J.J.A	1976 6 Month M-O	1977 3 Month J.J.A	1977 6 Month M-O
Mo. Valley					
025R	W	32	62	40	56
026R	E	39	66	32	47
Onawa					
027R	W	37	66	40	67
028R	E	55	73	39	69
Underwood					
029R	W	46	68	49	76
030R	E	55	81	50	76
Osceola					
031R	W	41	65	36	65
032R	E	38	65	47	76
Decatur Co.					
--					
034R	E	46	69	49	73
Pacific Jct.					
035R	W	38	59	40	69
036R	E	45	68	43	74
Clear Lake					
037R	W	44	70	42	68
038R	E	43	68	45	71
Linn Co.					
048R	E	35	65	37	72
049R	W	43	73	34	65
	Median	46	72	47	72
	Range	30-55	59-81	32-57	47-77

PART II

THE FEASIBILITY OF WIND-POWERED AERATION FOR REST AREA STABILIZATION PONDS

1. INTRODUCTION

1.1. Power from the Wind

When people started to use natural resources of energy for the production of mechanical power they turned to wind and to flowing water to augment their own power and that of their working animals. Thus, wind has propelled sailing ships for many centuries, and for stationary power, water mills and windmills were virtually the only source until the advent of the steam engine toward the end of the eighteenth century.

The advantages of wind energy are that (1) it does not deplete natural resources; (2) it is nonpolluting, making no demands upon the environment beyond a comparatively modest use of land area; and (3) it uses a cost-free fuel. These advantages must be weighed against the disadvantages: (1) the wind is a variable source of energy and thus not a reliable source of energy, and (2) the total system costs are high when a power storage system is included to overcome the first disadvantage.

There is no doubt that a considerable amount of energy can be obtained from wind power. The value of this power can be as high as a thousand kilowatt hours per year per square meter of the surface exposed to wind in windy areas. However, it is not always economically feasible to extract energy from the wind. When considering the use of wind for power purposes, the important questions are:

(1) Is there sufficient wind to be economically useful at the site considered?

(2) What annual amounts of wind energy can be expected?

(3) How is the wind distributed, in time, during the day, month, or year or even longer periods?

(4) What are the probable durations of very high wind speeds or of calm periods during any given period?

The two most important factors that enter into the question of the economic feasibility of wind power are (a) the annual mean wind speed and (b) the cost of a power generation by alternate methods. An annual mean wind speed that would be economically useful in an area where the cost of power generation is high might be quite different in another area where this cost is lower.

1.2. General Characteristics of the Wind

The availability of natural wind is a highly variable function of location and time. In general, both flat-plain regions and coastal regions experience winds that are characterized by positive velocity gradients with height above the ground surface. Mountainous regions and especially mountain crests experience, on the average, stronger surface winds than flat and coastal regions.

The daily wind pattern is highly variable. The wind speed and direction may change over wide ranges during a given day and from day to day. Daily periodic wind patterns recur in some areas. For example, some regions regularly experience higher wind velocities during the day than at night.

In marked contrast with the daily wind patterns, monthly average wind patterns vary only slightly throughout the year and from year to year. Therefore, it is much easier to predict monthly average speed and direction of the wind for a given area. In any given location, most of the monthly average wind velocities fall within 15 percent of the annual average [6]. The average wind velocity for the year would be expected to be more stable than the monthly averages.

The stability of the monthly average wind patterns and of the annual average wind pattern is of great importance in the utilization of wind power. The output reliability of a wind power system is directly dependent on the stability of the average wind patterns.

While the momentary velocity of the wind has an essential dynamic influence on a windmill and affects the work of the automatic adjusting system, the development of energy depends on the average velocity with respect to time and the area of the surface swept by the windmill.

The surface over which the wind flows affects the wind speed near that surface. A rough surface (such as buildings and trees) will produce more friction than a smooth surface (such as a lake). The greater the friction the more the wind speed is reduced near the surface. Figure 1 illustrates how the surface roughness affects the wind speed by means of a vertical wind speed profile. Table 1 gives the correction factors for extrapolating wind speed measurement at 30 ft from the ground surface to other heights over flat terrain of uniform roughness.

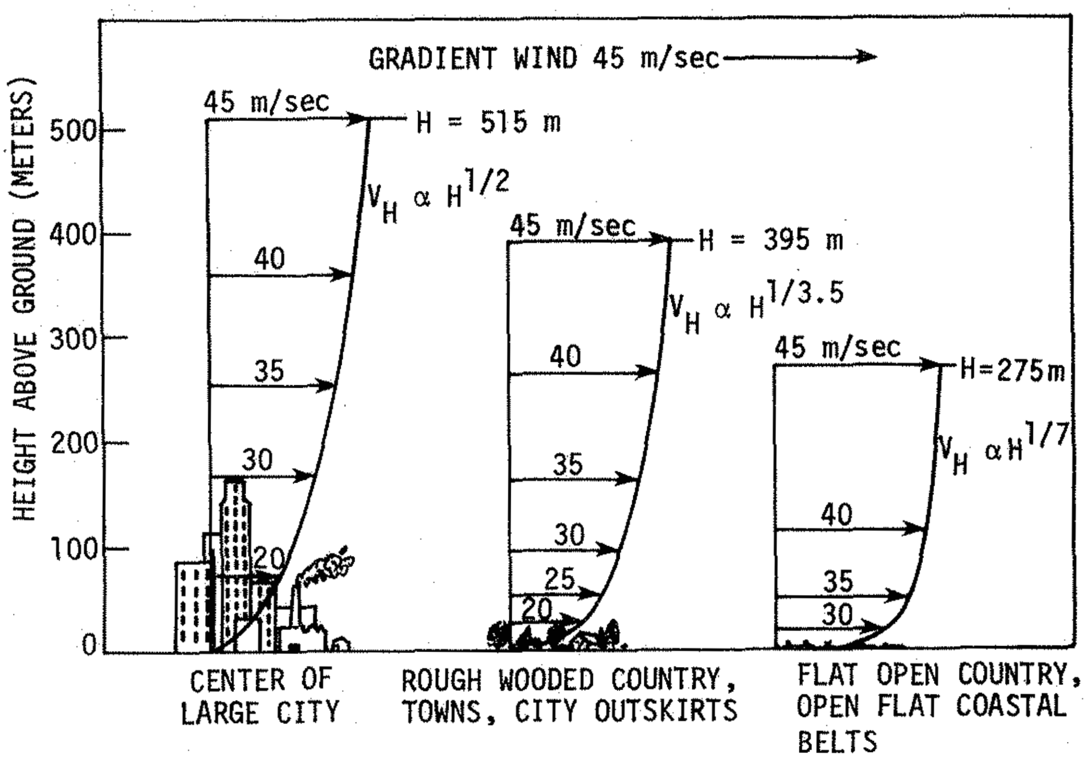


Fig. 1. Roughness of terrain lowers wind velocities near the ground surface. Most effective locations for wind power plants are in flat open country or on the crests of hills. [5]

Table 1. Extrapolation of the wind speed from 30 ft to other heights over flat terrain of uniform roughness [10].

Roughness Characteristic	20	40	60	80	100	120	140	160
Smooth surface: ocean, sand	0.94	1.04	1.10	1.15	1.18	1.21	1.24	1.26
Low grass or fallow ground	0.94	1.05	1.12	1.17	1.21	1.25	1.28	1.31
High grass or low row crops	0.93	1.05	1.13	1.19	1.24	1.28	1.32	1.35
Tall row crops or low woods	0.92	1.06	1.16	1.23	1.29	1.34	1.38	1.42
High woods with many trees	0.89	1.08	1.21	1.32	1.40	1.47	1.54	1.60
Suburbs, small towns	0.82	1.15	1.39	1.60	1.78	1.95	2.09	2.23

If the measured wind speed is not at the usual height of 30 ft, the wind speed at any other desired height can be estimated using the following equation.

$$\text{Estimated wind speed} = \frac{E}{k} \times s$$

E = the value for the height at which the wind speed is to be estimated from Table 1

k = the value for the height of measured wind speed from Table 1

s = measured wind speed.

2. THEORETICAL ASPECTS OF WIND POWER

2.1. Energy Content of Wind

Wind energy converting systems (WECS) convert the kinetic energy of air into work. The mass of air crossing the reference area A during a unit of time is

$$m = V_1 \times A \times \rho \qquad (\mathrm{Kg/s}) \tag{2}$$

and this can perform work at the rate of

$$L = m\frac{V_1^2}{2} = V_1^3 \times A \times \frac{\rho}{2} \qquad \frac{\mathrm{Kg}}{\mathrm{s}} \cdot \frac{\mathrm{m}^2}{\mathrm{s}^2} = \frac{\mathrm{Nm}}{\mathrm{s}} \tag{3}$$

Here, V_1 is the free stream air speed approaching the windmill perpendicular to the area, $\rho = \gamma/g$ is the density, and γ is the specific weight of air. The reference area A is the projected area for a rotor or the circular area swept out by the vanes for wind wheels as illustrated in Figure 2. Since air density ρ at a site normally varies only 10 percent or less during the year, the amount of power depends primarily on the reference area A and the wind speed V_1. Increasing rotor diameter or increasing the blade length will allow the wind energy converting systems to intercept more of the wind energy and thereby harness more power. Since the available power varies with the cube of the wind speed it is desirable to choose a site where the wind speed is higher than normal for the surrounding district.

The proportionality of the power to the cube of the wind speed is of fundamental importance. This forces the designer of a windmill

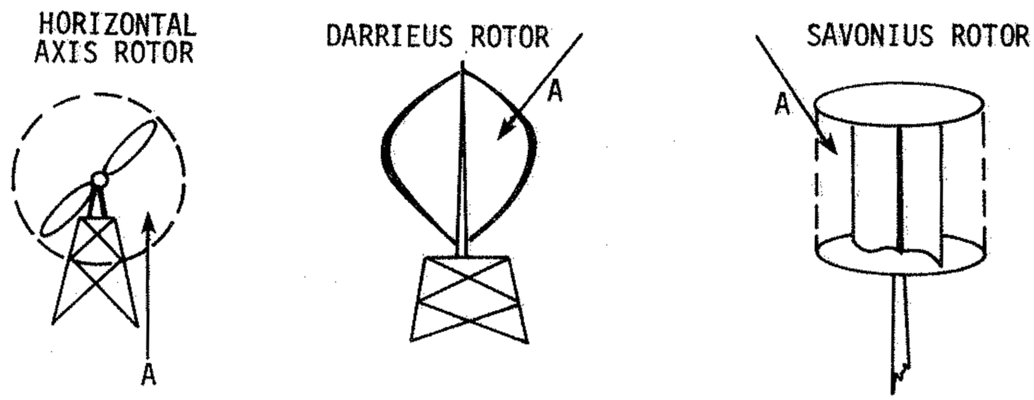


Fig. 2. Reference area "A" for different type of windmills.

to pose the question, "Up to what wind speed should the whole of the available power be used?" With this is associated another question: "What is the lowest wind speed at which an attempt should be made to extract power?" Or, using technical terms, "What should be respectively the rated (or design) wind speed and the cut in wind speed?" Power production begins at the cut in wind speed; power production levels off at the rated wind speed, and excess wind energy above the rated wind speed is not utilized.

It is important to select a reasonable value for rated wind speed because if the rated wind speed is unreasonably high the system will not be operating at rated output very much of the time because the frequency of occurrence of high speed wind is less; and if the rated wind speed is very low, most of the inherent energy in the wind will not be extracted.

The annual average wind speed and distribution or frequency of occurrence is clearly of great importance in assessing the energy potentialities of a site. The most essential information required when considering these potentialities is that relating to the annual duration of wind speed of different magnitudes. Wind speed measurements should thus determine hourly mean speed throughout the year. This can then be analyzed and displayed in the form of a velocity duration curve as shown in Fig. 3.

From the velocity duration curve it is possible to construct a power duration curve assuming the power is proportional to the velocity cubed (see Fig. 4).

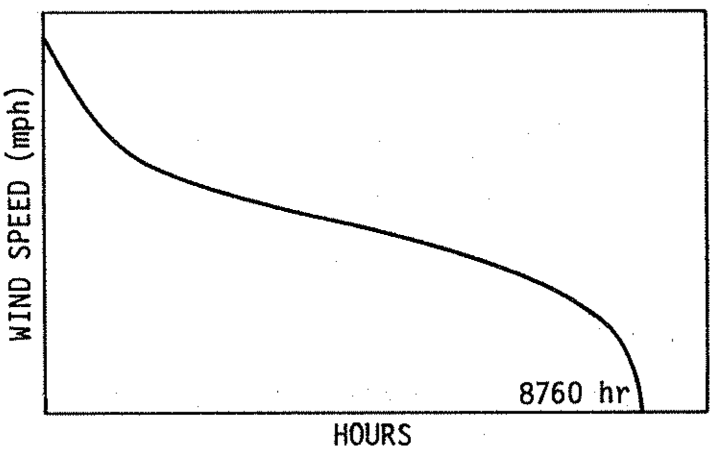


Fig. 3. Annual wind velocity duration curve.

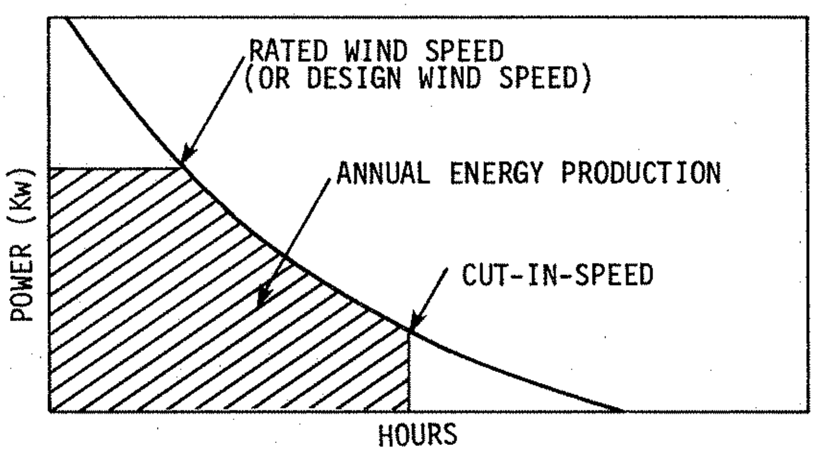


Fig. 4. Annual wind power duration curve.

2.2. Types of Wind Driven Machines

Wind-driven machines can be divided into two categories: (a) machines whose rotors move in a plane or planes perpendicular to the direction of the wind, and (b) machines having the effective moving surfaces of their rotors moving in the direction of the wind.

Figures 5 through 10 show diagrams of different types of windmills. Dutch plane-vane windmills, La Cour windmills, American farm windmills and wind turbine (propeller type) windmills fall into the first category. The Savonius rotor windmill and Darrieus vertical axis windmill are examples of the second type.

Comprehensive studies made in the United States and other countries have indicated that the wind turbine type of windmill, or rather more generally the horizontal axis type with radial blades, has the highest efficiency and is the most economical for power production [3]. But both climatic and economic conditions vary so greatly in places where wind power could be utilized that it would be wrong to dismiss all the other types as inferior.

The actual operating data for windmills are expressed by means of dimensionless coefficients:

$$\text{Power} \quad L = C_\ell \times A \times V_1^3 \times \frac{\rho}{2} \quad \text{in kw} \tag{4}$$

$$\text{Torque} \quad M_r = C_d \times R \times A \times V_1^2 \times \frac{\rho}{2} \quad \text{in } \frac{\text{Kg m}^2}{\text{s}^2} = \text{Nm} \tag{5}$$

$$\text{Axial Thrust} \quad S = C_w \, A \times V_1^2 \times \frac{\rho}{2} \quad \text{in } \frac{\text{Kg m}}{\text{s}^2} = \text{N} \tag{6}$$

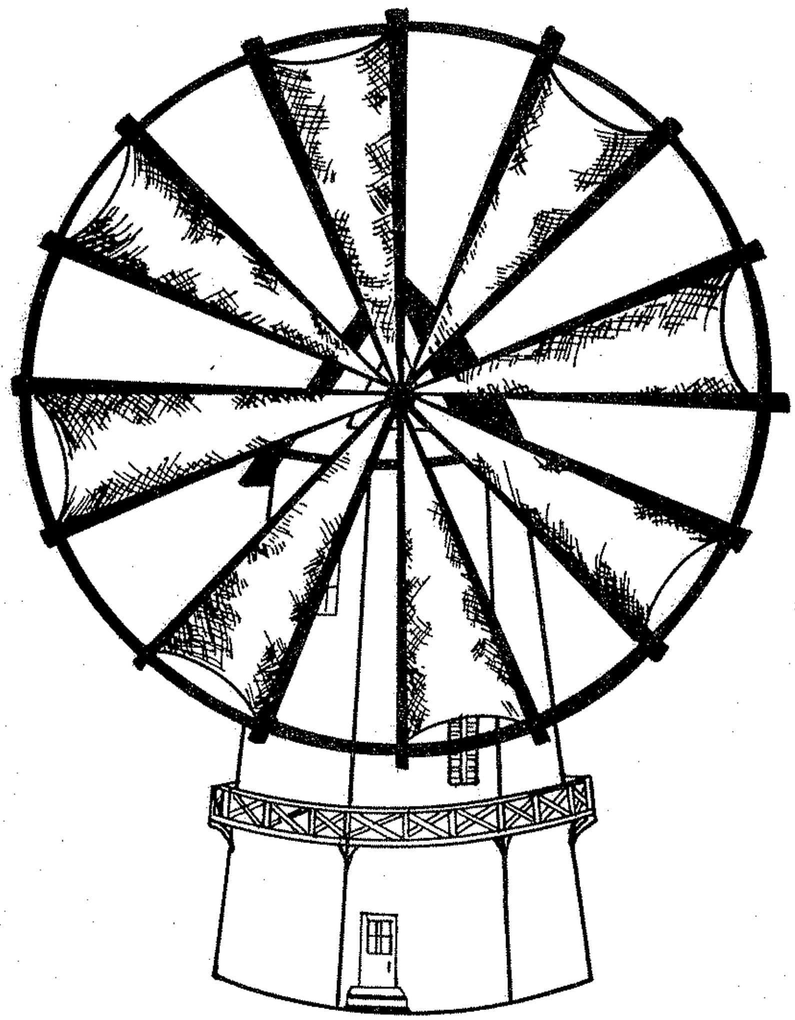


Fig. 5. Schematic diagram of an early eighteenth century Dutch plane-vane windmill.

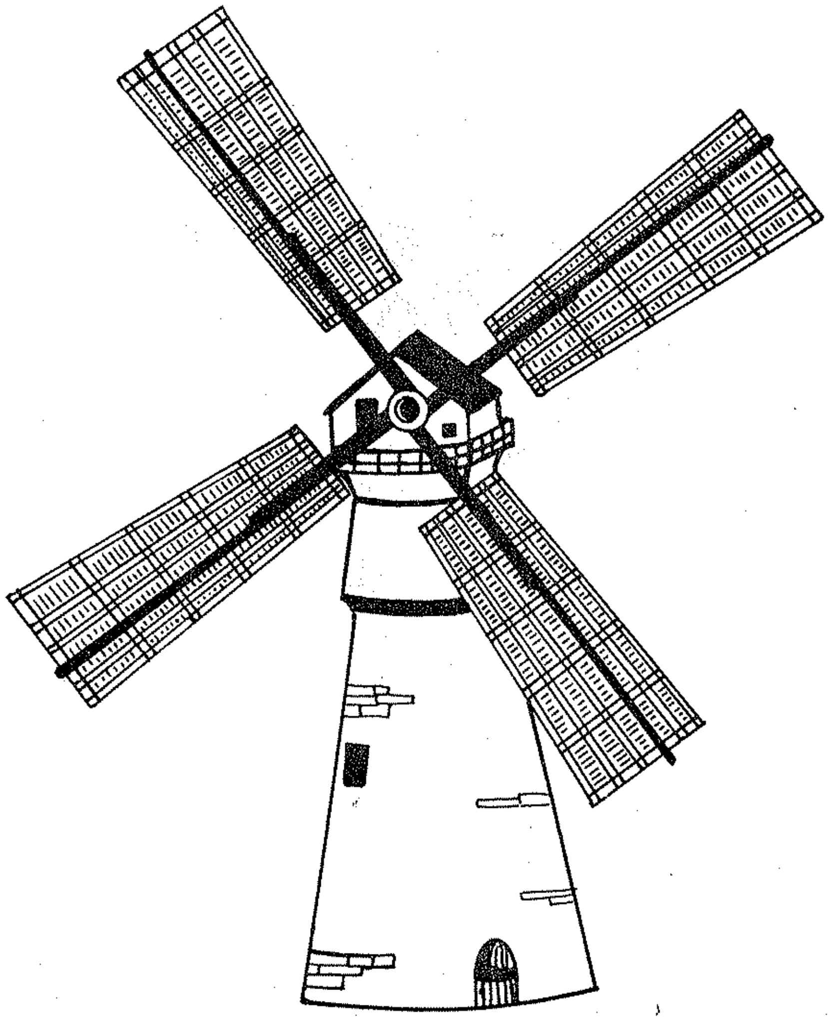


Fig. 6. Schematic diagram of a LaCour windmill.

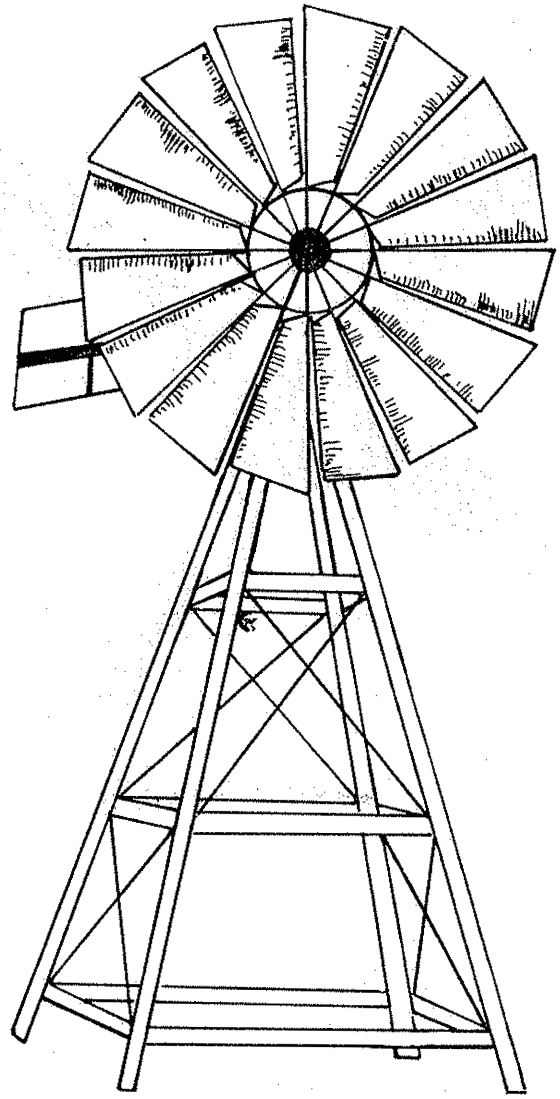


Fig. 7. Schematic diagram of an early American windmill.

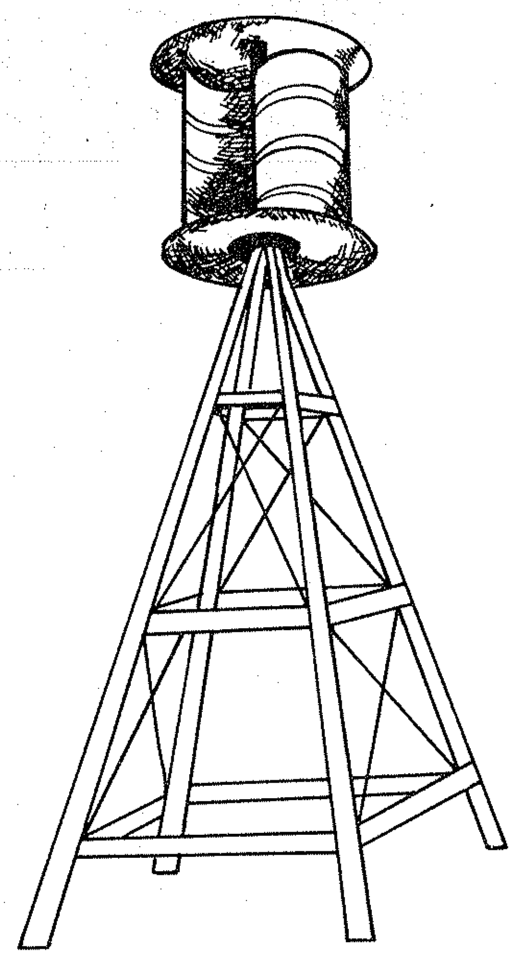


Fig. 8. Schematic diagram of a Savonius rotor windmill design.

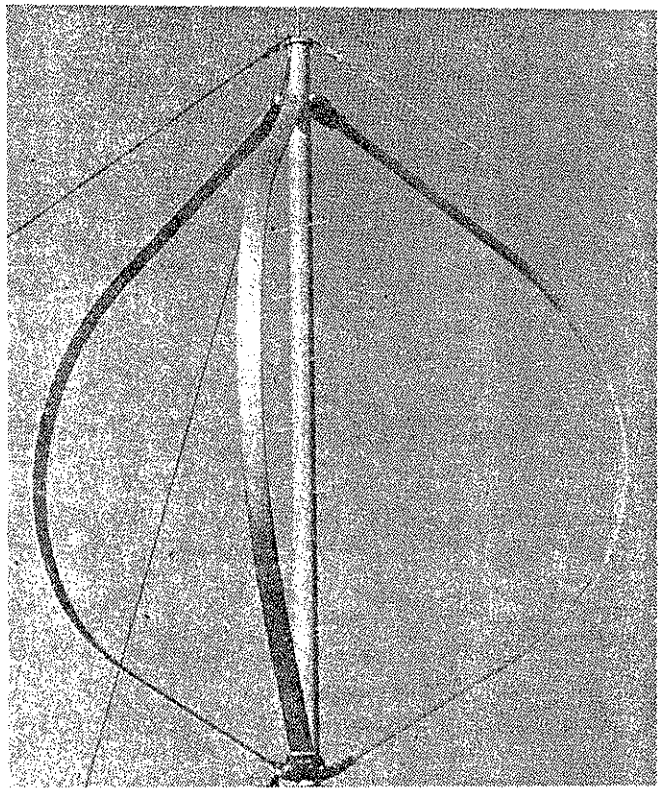


Fig. 9. Darrieus vertical axis windmill.

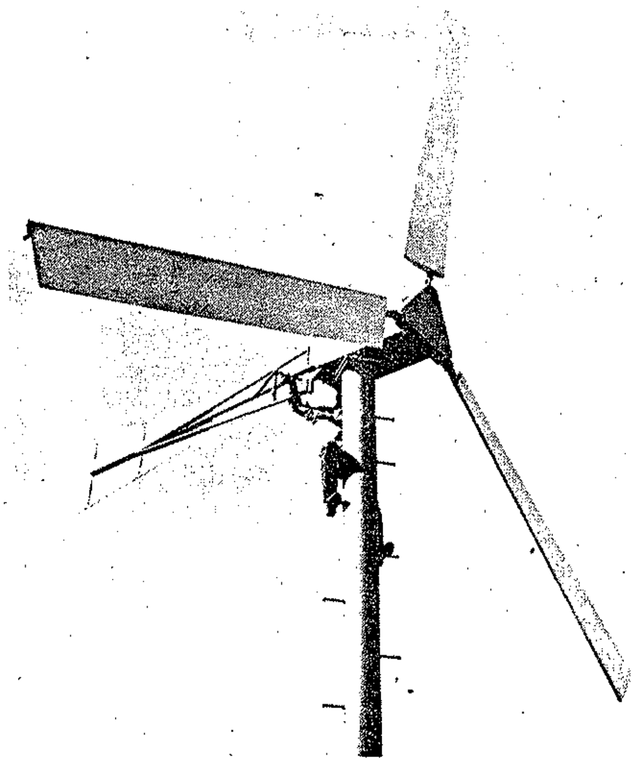


Fig. 10. Wind turbine.

$$\text{Revolutions} \quad n = \frac{30\ \lambda_o\ V_1}{\pi R} \quad \text{in rpm} \tag{7}$$

Where C_ℓ, C_d, C_w, are power, torque and thrust coefficients respectively and are dimensionless;

λ_o = u/V_1 is the tip-to-wind-speed ratio;

u = $\pi Rn/30$ is the tip speed, and

R = the radius to the tip of the blade.

2.3. The Power Coefficient

The power coefficient, which is in effect equal to the efficiency of the windmill, can never be 1 even under ideal conditions.

A. Betz, of the Institute of Gottingen, Sweden, in 1927 showed by applying simple momentum theory to the horizontal axis windmill that the maximum fraction of the power in the wind that could be extracted by an ideal windmill was 16/27 or 0.593 (see Fig. 11).

This theoretical efficiency of Betz can be developed as follows:

Let

V_1 be the velocity of air upstream of the windmill,

V_2 be the velocity of air downstream of the windmill,

V be velocity of air at the windmill,

$\dot{m}$ be the mass flow rate of air per unit time.

$$\text{Change of momentum} = \dot{m}(V_2 - V_1) \tag{8}$$

$$\therefore \text{Work done on the windmill} = \dot{m}V(V_1 - V_2) \tag{9}$$

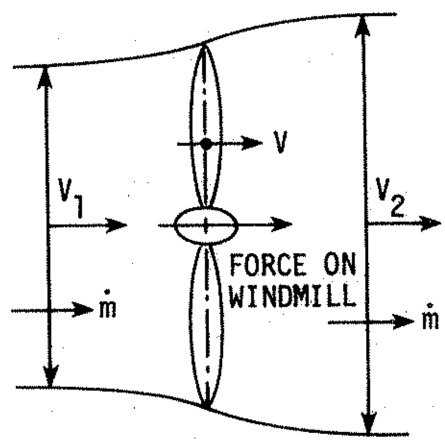


Fig. 11. Schematic diagram of a windmill extracting energy from wind.

Change of kinetic energy of the air stream = $(1/2)\dot{m}(V_1^2 - V_2^2)$ (10)

Work done = change of kinetic energy

$$\dot{m}V(V_1 - V_2) = (1/2)\dot{m}(V_1^2 - V_2^2)$$

$$V = \frac{V_1 + V_2}{2} \tag{11}$$

$$\text{Work done} \quad L = \dot{m}\left(\frac{V_1 + V_2}{2}\right)(V_1 - V_2) \tag{12}$$

$$\dot{m} = \rho \times A \times V \tag{13}$$

Where ρ is the density of air and A is the cross sectional area through which air flows.

$$L = \rho\ A\ V\left(\frac{V_1 + V_2}{2}\right)(V_1 - V_2) \tag{14}$$

substituting for V from (11)

$$= \rho\ A\left(\frac{V_1 + V_2}{2}\right)^2 (V_1 - V_2) \tag{15}$$

$$\text{Let } \frac{V_2}{V_1} = \alpha \tag{16}$$

$$L = \frac{\rho A V_1^3}{4}(1 + \alpha)^2 (1 - \alpha) \tag{17}$$

$$\frac{dL}{d\alpha} = \frac{\rho AV_1^3}{4} \{(1 - \alpha) \ (2) \ (1 + \alpha) + (1 + \alpha)^2 \ (-1)\}$$

$$\frac{dL}{d\alpha} = \frac{\rho AV_1^3}{4} \{2(1 - \alpha 2) - (1 + \alpha)^2\}$$

To find the maximum or minimum power set

$$\frac{dL}{d\alpha} = 0$$

which leads to $\alpha = 1/3$. To determine if it is a maximum, take the second derivative:

$$\frac{d^2L}{d\alpha^2} = \frac{\rho AV_1^3}{4} \{-4\alpha - 2(1 + \alpha)\} = -\rho AV_1^3 \ (6\alpha + 2)$$

when $\alpha = 1/3$

$$\frac{d^2L}{d\alpha^2} < 0$$

$\therefore$ L is maximum when $\alpha = 1/3$. Substituting $\alpha = 1/3$ back into equation (17) yields the maximum power output.

$$\text{Maximum output power} = \frac{\rho AV_1^3}{4} \left(\frac{16}{9} \times \frac{2}{3}\right) = \frac{8\rho AV_1^3}{27} \tag{18}$$

Dividing equation (18) by equation (3) gives the efficiency

$$\begin{aligned} \text{Efficiency of an ideal windmill} &= \frac{8\rho AV_1^3}{27} \times \frac{1}{(1/2)\rho AV_1^3} \\ &= \frac{16}{27} = 0.593 \end{aligned} \tag{19}$$

But because of aerodynamic imperfections in any practical machine and mechanical and electrical losses the actual efficiency of windmills is much less than 0.593.

Table 2 shows the typical efficiency and tip-to-wind-speed ratio for different types of windmills [6].

Table 2. Typical efficiency and tip-to-wind-speed ratios for different types of windmills.

Type of Windmill	Efficiency Range % (or Power Coefficient)	Typical Tip-to-wind-speed Ratio
Dutch (plane-vane)	5-10	0.5-1.0
La Cour (four-vane)	20-22	2.3-2.5
American Farm (multivane)	15-30	1.0-2.0
S-rotor (Savonius)	30-35	0.7-1.7
Wind Turbine (propeller type)	35-40	5.0-10.0

2.4. The Torque Coefficient

The torque coefficient can be shown as

$$C_d = \frac{C_\ell}{\lambda_o} \tag{20}$$

Where C_ℓ is the power coefficient

λ_o is the tip-to-wind-speed ratio.

The zero torque coefficient C_{do} with the wheel stopped is a function of the type of windmill, angular variations of the blades, the blade profile and outline configurations.

For example, the blades of a high-speed windmill are almost parallel to the plane of rotation and the flow separates along almost the entire blade when the wheel is stationary. As a result of this, the torque coefficient for high-speed wheels is very small at the start-up, (i.e., at low-starting torque). The start-up properties of high-speed wheels can be improved by altering the blade orientation either automatically or manually so that the flow adheres to the profile along most of the blade radius.

2.5. Relative Advantages of Different Machines

Figure 12 shows the power and torque coefficients of windmills of different tip-to-wind-speed ratios. Comparison of C_ℓ and C_d curves of windmills of different designs shows the superiority of low-speed windmills (Savonius, multivane, La Cour) in providing better starting torque, and the superiority of high-speed wheels (Aeroturbine) in providing more power and higher rotational speed.

The two vertical axis type windmills, the Savonius rotor and the Darrieus vertical axis, were developed in the early part of the twentieth century. The Savonius rotor was formed by cutting a cylinder into two semicylindrical surfaces, moving these surfaces sideways along the cutting plane to form a rotor with cross-section in the form of the letter S, placing a shaft in the center of the rotor and

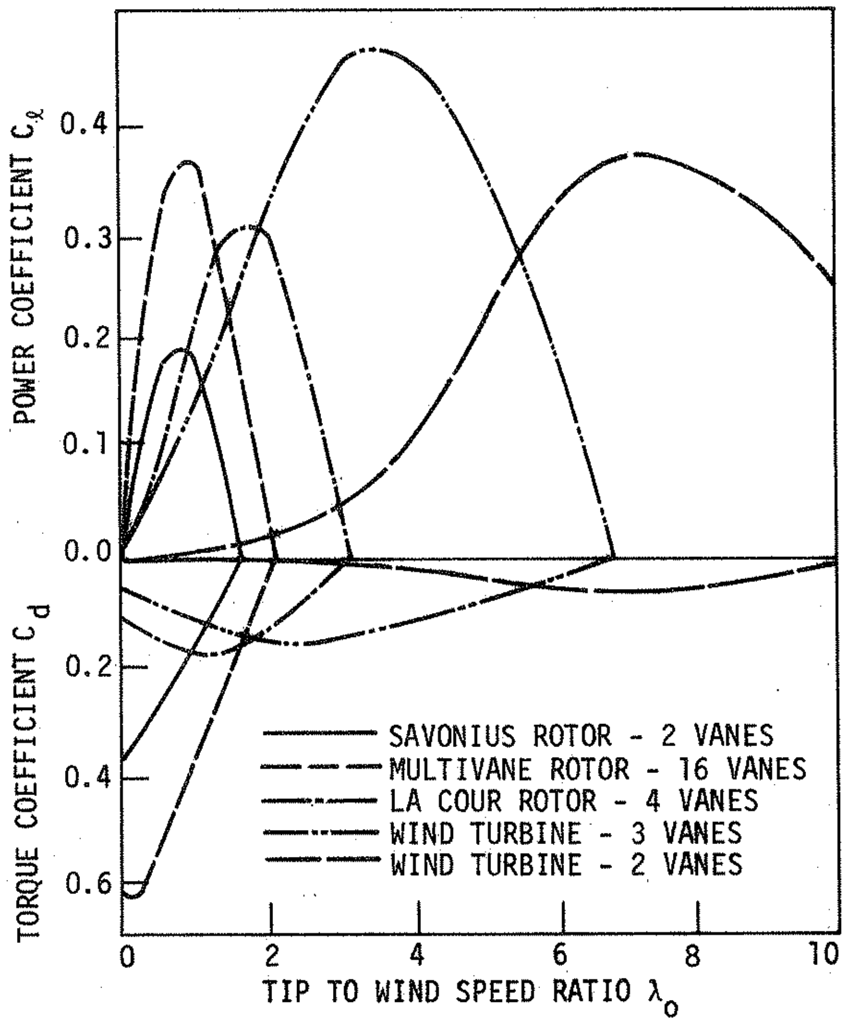


Fig. 12. Power and torque coefficients of windmills with different designs and tip to wind speed ratios [4].

closing the end surfaces with circular end plates. This improved design was able to produce an efficiency of about 30 percent which was significantly higher than was obtainable with other types of vertical axis windmills in operation at that time [7]. Savonius attributed this improvement to asymmetric or magnus effects as shown in Figure 13. The disadvantage of the Savonius rotor is that it is inefficient per unit weight. To produce 1000 kw in a 30 mph wind a Savonius rotor requires about 30 times as much metal as a two-bladed turbine.

The Darrieus vertical axis windmill shown in Fig. 10 consists of two or three thin airfoils with one end of the foil mounted on the lower end of a vertical shaft and other end mounted on the upper end of the same shaft. This design uses comparatively less metal. The disadvantages of a Darrieus rotor are that the rotation will not begin with a wind velocity less than 10 mph and the aerodynamics of this rotor are not simple.

Another design for extracting power from the wind became popular toward the end of World War I. This design is called a wind turbine (see figure 10). Many different designs of wind turbines using two or more blades have since been considered. The two- and possibly the three-bladed designs appear to be the most suitable for electric power generation. In general higher rotational speeds may be obtained with a two-bladed design. However, addition of more blades may increase the starting torque, reduce the tip losses and improve the smoothness of operation.

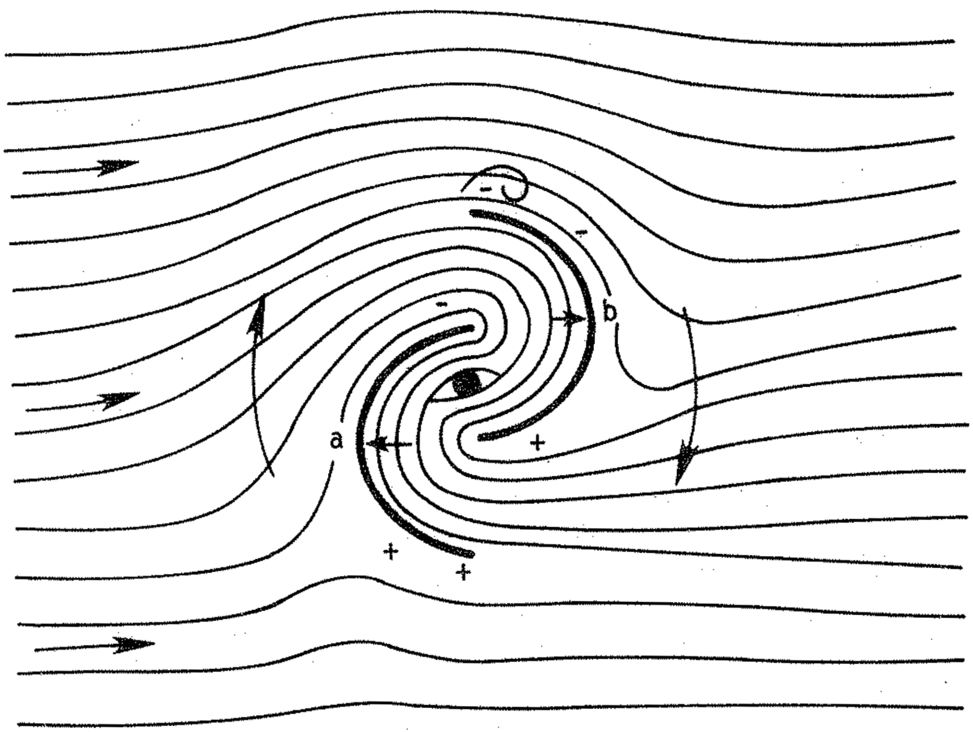


Fig. 13. Air streaming and pressure differences around an S-Rotor.[7]

(Note the flow through the central air passage, the smooth streaming, and the absence of a vacuum at the back of the advancing vane.)

2.6. Power Calculations

An attempt is made in the following pages to verify the power potential of different types of windmills by comparing the theoretical output calculated using the wind velocity, rotor area, and air density, with that supplied by the respective manufacturer. If a fairly consistent value for the efficiency is obtained, the prediction for that machine is deemed to be satisfactory. Descriptive information about each machine is presented in the appendix, Section 8.

Specimen Calculations

Type: Typical American Farm Windmill

Mfgr: Dempster, Model: 10 ft

Total Elevation (water lift) = 119 ft

Flow rate = 300 gallons per hour (gph)

Wind speed = 15 miles/hr (mph)

$$\text{Pumping power output} = 119 \text{ ft} \times 300 \text{ gph} \times 8.34 \frac{\text{lb}}{\text{gal}} \times \frac{1 \text{ hr}}{3600 \text{ sec}} \times \frac{1 \text{ hp}}{550 \text{ ft lb/sec}} = 0.15 \text{ hp}$$

Windmill Power Calculation

Air specific weight at 14.7 psi and 30 $^{\circ}$C = 0.0725 lb_f/ft^3

$$\text{mass density } \rho = \frac{0.0725}{32.2} = 2.25 \times 10^{-3} \text{ slug/ft}^3$$

$$= 2.25 \times 10^{-3} \text{ lb}_f \text{ sec}^2/\text{ft}^4$$

$$\text{Theoretical wind power} = 1/2 \, \rho A V^3$$

$$= (1/2) \times 2.25 \times 10^{-3} \times \frac{\pi}{4} \times (10)^2 \times \left(15 \text{ mph} \times \frac{22 \text{ ft/sec}}{15 \text{ mph}}\right)^3$$

$$= 940.8 \frac{lb_f \ ft}{sec}$$

$$= \frac{940.8}{550} = 1.71 \text{ hp}$$

Efficiency of windmill and pumping machine combined

$$= \frac{0.15}{1.71} \times 100 = 8.77\%$$

Calculated Efficiencies of Different Windmills

Dempster Windmill (see Appendix)

Type: American Farm Windmill

Dempster Industries, Inc.
P. O. Box 848
Beatrice, NB 68310

Rotor Diameter ft	Elevation* (Water Lift) ft	Water Flow* gph	Pumping horsepower	Wind Power @ 15 mph hp	Efficiency %
6	120	115	0.058	0.616	9.4
	54	248	0.056		9.1
8	172	173	0.125	1.095	11.4
	77	370	0.120		10.9
10	256	140	0.150	1.711	8.8
	119	300	0.150		8.8
12	388	180	0.294	2.463	11.9
	173	390	0.284		11.5
14	580	159	0.388	3.353	11.6
	260	334	0.365		10.9

*From catalog information in the Appendix.

Comments: Efficiency ranges from 8.8 to 11.9 percent. This includes the efficiency of the windmill and the pumping mechanism. If the efficiency of the pumping mechanism is assumed to be 0.7, then the efficiency of the windmill alone could be expected to be about 1.5 times the above values. The reported efficiency for multivaned windmills ranges from 15-30 percent (Table 2), so the results above seem quite reasonable and consistent.

Heller - Allen Windmills (see Appendix)

Type: American Farm Windmill

Heller - Allen Company
Perry and Oakwood Streets
Napoleon, Ohio 43545

Rotor Diameter ft	Elevation (Water Lift) ft	Water Flow gph	Pumping horsepower	Wind Power @ 15 mph hp	Efficiency %
6	25	350	0.037	0.616	6.0
	125	120	0.063		10.22
8	25	900	0.095		8.67
	100	250	0.105	1.095	9.59
10	25	1250	0.132		7.72
	75	475	0.15	1.71	8.77
12	25	2400	0.253		10.27
	100	600	0.253	2.463	10.27

Comments: Since the efficiencies include both the windmill and the pumping unit, the calculated efficiences fall within reasonable range except for one low 6% value.

WTG Systems (see Appendix)

Type: Wind Turbine

WTG Energy Systems
Box 87 LaSalle Street
Angola, NY 14006

Rotor Diameter = 80 ft

Cut-in wind speed = 8 mph

Wind Speed mph	Power Output* kw	Theoretical Wind Power kw	Efficiency %
19	50	166	30.1
22	75	258	29.1
23.5	100	314	31.8
27	150	476	31.5

*From catalog information in the Appendix.

Comments: The calculated efficiencies fall within reasonable range, and the values are consistent with those reported in the literature for propeller type windmills.

Storm Master (see Appendix)

Type: Wind Turbine

Wind Power System Inc.
P. O. Box 17323
San Olego, Ca. 92117

Rotor Diameter = 32.8 ft

Cut-in Wind Speed = 8 mph

Rated output = 6000 watts at 18 mph

Wind Speed mph	Power Output kw	Theoretical Wind Power kw	Efficiency %
10	1.0	4.06	24.6
15	3.7	13.7	27.0
20	8	32.5	24.6

Comments: The calculated efficiency values are fairly consistent but they are slightly lower than the efficiency value reported in the literature for this type of windmill.

Aero Power Systems Inc. (see Appendix)

Type: Wind Turbine

Model SL 1500
2398 Fourth Street
Berkely, CA 94710

Rotor Diameter = 12

Cut-in Wind Speed = 6 mph

Rated Wind Speed = 25 mph

Wind Speed mph	Power Output kw	Theoretical Wind Power kw	Efficiency %
10	0.25	0.73	34
15	0.90	2.46	37
20	2.0	5.84	34
25	4.0	11.4	35

Comments: The calculated efficiency values are very consistent and within the allowable range for turbine type windmills.

2.7. Potential Power of S-Type Rotors

The initial interest in the use of wind power aeration of the rest area ponds was stimulated by the availability of some small inexpensive S-Type floating wind driven devices used primarily to prevent total ice cover in lakes and therefore winter fish kill. The foregoing power calculations have demonstrated that the theoretical equations for power generation agree with the manufacturers' data for those machines. Therefore, it was of interest to calculate the potential power generated from the small S-type rotors.

Based on typical summer BOD loads, it is estimated that about 1 hp would be required to run a conventional floating aerator to supply the oxygen needs of the primary pond. It will thus be of interest whether these small floating S-type rotors can come close to producing 1 hp.

Potential Power S-Type Rotors (see Appendix)

Type: somewhat like Savonius rotor.

Pondmaster Econo 271 370, & 672

Wapler Manufacturing Co., Galena, Kansas 66739

The following calculations are based on the largest model, the Econo Model 672, and the following assumptions: The average wind speed during the months of June, July and August is 9 mph. However, this calculation is made for a design wind speed of 15 mph. Rotor has a diameter of 2 ft and is 7 ft in height. (The diameter is estimated from the catalog drawings.) Available wind power:

Projected area of the rotor $= 7.0 \text{ ft} \times 2 \text{ ft} = 14 \text{ ft}^2$

Available power

$$= 1/2\ \rho \times A \times V^3$$

$$= 0.5 \times 2.25 \times 10^{-3} \times 14$$

$$\times \left(15 \times \frac{22}{15}\right)^3 \frac{\text{lb}_f \text{ ft}}{\text{sec}}$$

$$= 0.30 \text{ hp}$$

Assume an efficiency of 30 percent even though the Pondmasters are not designed in the manner recommended by Savonius [7] and thus are probably less efficient than the Savonius rotor.

Output power $= 0.3 \times 0.30$

$= 0.09$ hp

Therefore, the power potential of this type of rotor at practical summer design wind speeds is far below the needed 1 hp. Furthermore, the wind speed used in this calculation was not corrected for the height of installation which is only a few feet above the surface of the pond.

3. *ECONOMIC DESIGN OF WIND-POWER SYSTEMS*

The cost of wind-power systems is very difficult to analyze because of the many variables that must be assessed. Given a wind-power system design with a known investment cost, the cost per unit of output is a function of the mean annual wind speed at the site and of the fluctuation of the actual wind speed from the annual mean. The results of a study made by a team of United Nations investigators [8] indicate the average effects of these variables. The calculations presented in the following tables are for the propeller or turbine type windmills. However the same general trend will hold true for the other types of windmills.

Table 3 shows the total wind-power system cost (i.e., capital cost), the system power capacity, and the cost per unit of capacity as a function of design or rated wind speed.

Table 3. Relative wind-power system cost, power capacities and costs per unit of capacity as functions of design wind speed [8].

Design Wind Speed mph	Relative Sys. Cost (a)	Relative System Power Capacity (b)	Relative cost per Unit of Capacity (a)/(b)
35	1.00	1.00	1.00
30	0.86	0.63	1.36
25	0.71	0.36	1.97
20	0.57	0.19	3.00
15	0.43	0.08	5.37

From Table 3, the economic importance of designing a wind-power system for the proper wind speed can be seen. The cost per unit of output of a wind power system designed for 15 mph wind is 5.37 times the corresponding cost of the system designed for 35 mph wind. However this rise in cost is not the only important criterion for the economy of energy generation by a wind-power system. Reduction in design wind speed affects the achievable annual energy output per unit of installed capacity, which is called the specific output and is measured in kwh generated per kw installed. The specific output is an important performance parameter for a wind-power system. *Table 4 shows the relative specific output as* a function of the design wind speed for sites with different annual mean wind speeds.

Table 4. Relative specific output of wind-power systems as a function of the design wind speed for sites with different annual mean wind speeds [8].

Design Wind Speed mph	Relative specific outputs for given annual mean wind speeds (annual kwh generated per kw installed).		
	(a) 10 mph	(b) 15 mph	(c) 20 mph
35	1.0	8.0	16.1
30	2.9	12.4	22.2
25	6.3	19.1	30.1
20	12.4	27.2	39.3
15	24.2	37.8	48.1

By referring to Tables 3 and 4 it can be seen that although reduction in design wind speed from 35 mph to 15 mph increases the system cost per kw capacity more than fivefold, the specific output is simultaneously increased more than 24 times if the annual mean wind speed at the site is actually 10 mph.

Using the information available in Tables 3 and 4, Table 5 has been computed to show the relative costs per unit of electrical energy output for wind-power systems as functions of the annual wind speed and the annual capital charges for sites with different annual mean wind speeds.

For example, if the design speed is 25 mph, from Table 3 the relative cost per unit of capacity = 1.97. From Table 4, at 20 mph annual mean wind speed, the relative specific output = 30.1 kwh/kw. Therefore, the relative cost per unit of electrical energy output for given annual mean wind speed =

$$\frac{1.97}{30.1} = 0.065$$

From Table 5 it can be seen that, when an installation site experiences a mean wind speed of 15 mph, the optimum value for design wind speed is about 25 mph. For installation with low annual mean wind speed of about 10 mph, the design wind speed should be from 15 to 20 mph.

Table 5. Relative costs per unit of electrical energy output for windpower systems as functions of the design wind speed and the annual capital charges for sites with different annual mean wind speeds.

Design Wind Speed mph	Relative Annual Capital Charges[1]	Relative costs per unit of electrical energy output for given annual mean wind speeds		
		10 mph	15 mph	20 mph
35	1.00	1.000	0.125	0.062
30	1.36	0.469	0.110	0.061
25	1.97	0.313	0.103	0.065
20	3.00	0.242	0.110	0.076
15	5.37	0.222	0.142	0.112

1. The same percentage of annual capital charges is assumed in all cases.

4. APPLICATION OF WIND POWER TO WASTEWATER TREATMENT

4.1. Notes on Iowa Wind Patterns

The wind speeds across Iowa show small differences across the State [11]. The average wind speed variations are within 0.6 mph with slower wind speeds occurring along the eastern portion of the state. A general wind speed profile for the state excluding the NE Mississippi Valley area is presented in Figure 14.

At 20 ft above ground level, average monthly wind speeds vary from 7.8 miles per hour in August to 12.4 mph in April. March and April experience the highest monthly average speeds while July and August have the lowest monthly average speeds.

Diurnal wind profiles for Iowa locations indicate that wind speeds are lowest in the early morning hours between 1:00 and 6:00 a.m. Central Standard Time, and reach peak speeds between 1:00 and 3:00 p.m. CST.

Figure 15 shows the wind duration curve for Des Moines Airport for years 1955 and 1964 [9]. The two curves follow closely together except at the speed ranges between 20 and 30 mph. Either of these curves could be used to calculate the average annual wind energy potential for the site, as shown in Figures 3 and 4.

4.2. Previous use of wind energy for aeration of wastewaters.

The review of literature revealed only two studies reporting the use of wind energy for the aeration of wastewaters.

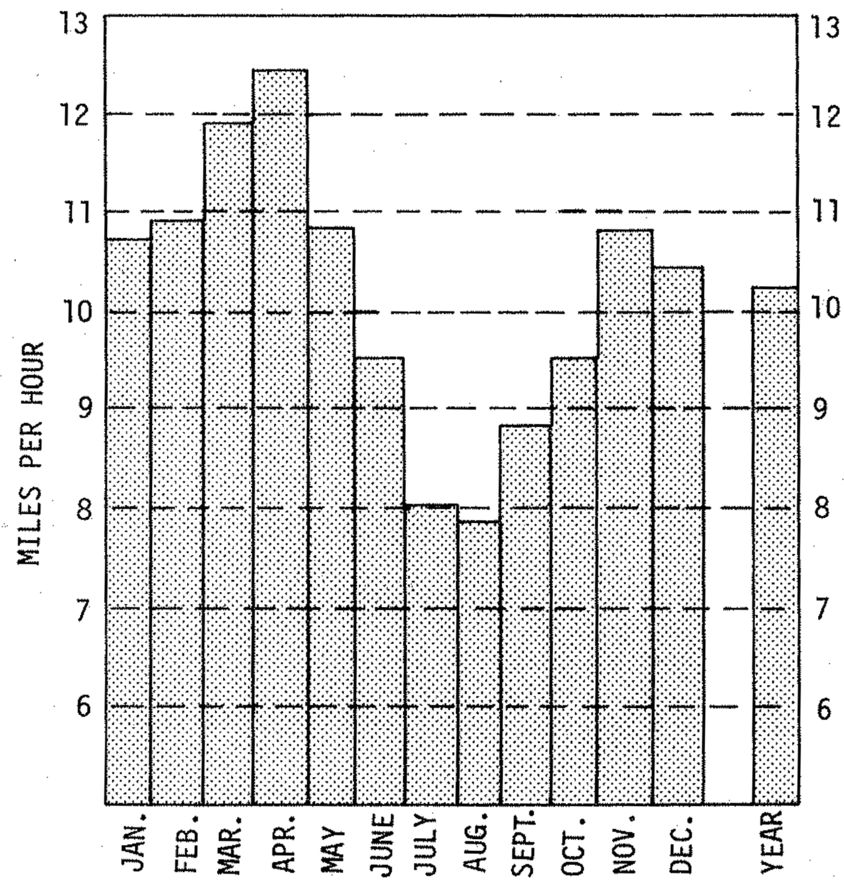


Fig. 14. Iowa composite average wind profile adjusted to 20 feet above the ground. [11]

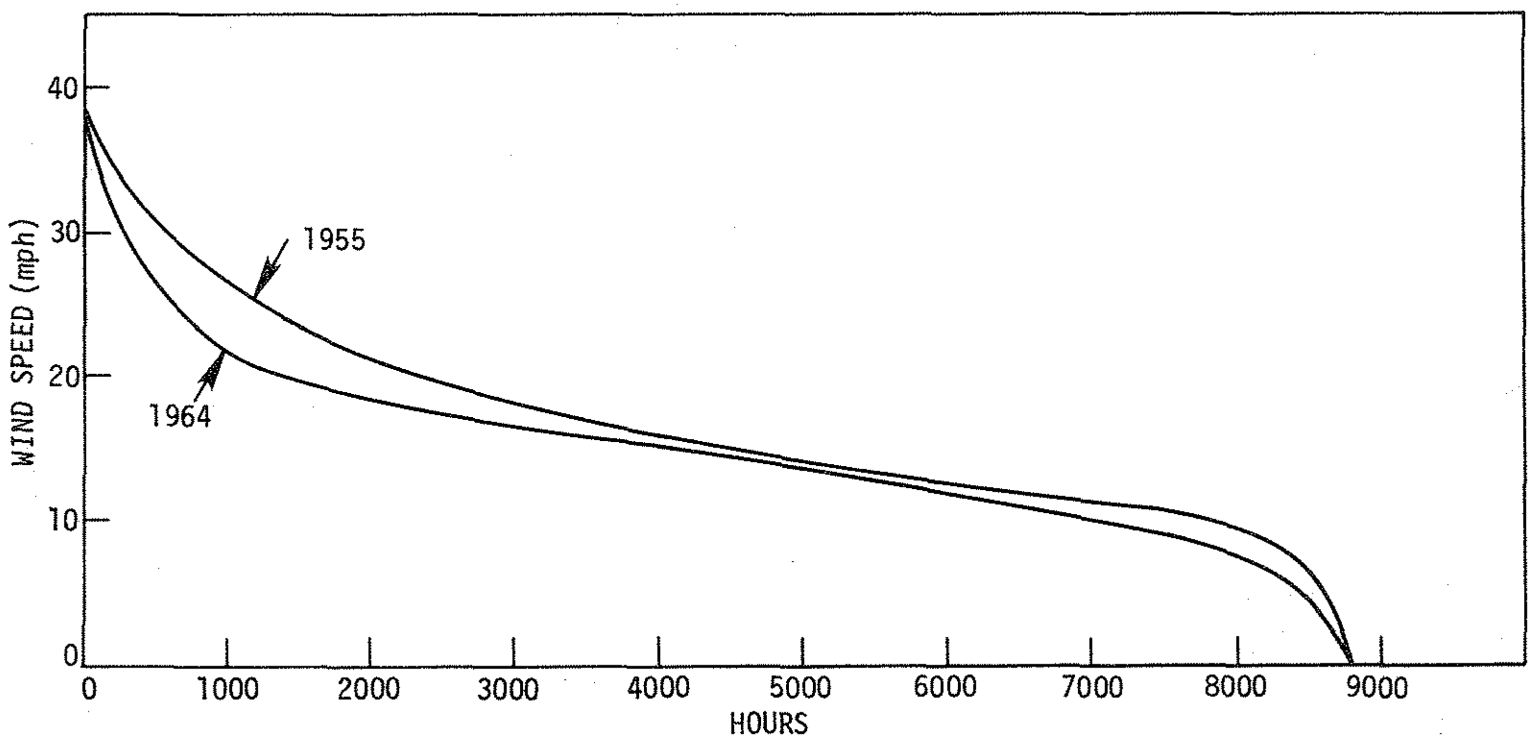


Fig. 15. Wind duration curve for Des Moines Airport. Wind speed recorded at 20 ft above ground level.

In the study conducted jointly by Colorado Division of Wildlife and Colorado State University [1] the wind turbine was used to drive an air compressor through a direct mechanical linkage. The compressed air was injected into the bottom of the sewage lagoon to improve the biological waste treatment efficiency. The authors of this study suggested that if matching of the load characteristics of the wind turbine and the compressor were the prime requisite, then a centrifugal blower should be considered. The centrifugal blower develops pressure proportional to the speed squared and requires power proportional to the speed cubed just as the power potential of the windmill. However, if the compressed air is to be injected at the bottom of the sewage lagoon, the blower must develop sufficient pressure head to overcome the static head of liquid column above the air outlet and frictional head loss in the supply line. A contrifugal blower may require a very high speed to overcome these pressure heads. In contrast, a positive displacement type compressor can overcome this resistance at low wind velocities and slower shaft rmp. But the positive displacement type compressor will require high starting torque. Therefore, a windmill with high starting torque should be considered if a direct mechanical linkage between air turbine and positive displacement air compressor is contemplated.

In the other study, conducted in Quebec, Canada, the aeration system that was under study is shown in Fig. 16. [2]. It consisted of M units (consisting of a wind turbine, an alternator with solid state rectifier and regulator) operating in parallel with a battery system to provide the necessary starting capacity. The load consisted of an air compressor driven by a DC motor. With this setup it can be assumed that

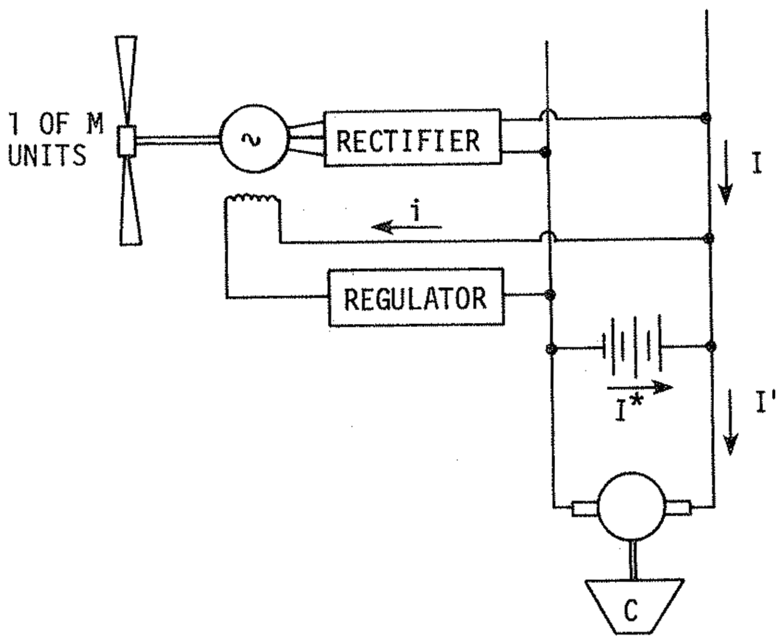


Fig. 16. Schematic representation of aeration system.

load conditions are invariable, i.e., that the compressor supplies a constant volumetric flow rate Q at a fixed head (Δp). The authors concluded that the proposed concept is realistic and possible with existing technology. However, they said that the performance of a wind-driven turbine mechanically coupled to a compressor should be evaluated to ensure the best possible combination of components.

4.3. Use of Wind Power for Aerating Rest Area Stabilization Ponds in Iowa

Given the general description of wind power presented in the preceeding pages, an attempt will be made now to look into the feasibility of using wind power for aerating the rest area stabilization ponds in Iowa.

The peak loads on these ponds occur in the months of June, July and August. Therefore it is during this period that much aeration will be required. The monthly average wind speeds for the state of Iowa, presented in Figure 14, show that July and August have the least wind velocity. The average wind speed for August is about 8 mph. Therefore, windmills with cut-in speed less than 7 mph should be chosen.

The annual average wind speed for the state of Iowa is about 10 mph. Therefore, as suggested by the Table 5, the design wind speed should be around 15 mph for economical operation.

The wind pattern in the State of Iowa is in direct opposition to what would be desired for good operation of a stabilization pond using a directly driven wind powered aeration device. The first detrimental factor is the fact that the annual monthly wind speeds for July and August are lowest compared with other months of the year. It is during this period, i.e., June, July and August, that the ponds receive summer peak loads. Therefore, need for aeration would be greatest in these months. But as the wind speed is low, not much power will be produced by the windmill during this period.

The second factor that acts in opposition to what is desired is the diurnal variation in the daily wind speeds. The wind speeds are lowest between 1:00 and 6:00 a.m. and reach their peak between 1:00 and 3:00 p.m. In the late afternoons the algae found in the ponds are highly productive and the dissolved oxygen in the pond is fairly high. Therefore, aeration is not as critical in the afternoon as it is at night or in the early morning hours. For this reason, if the windmill does not have any provision for storage of energy, the wind power will be used inefficiently and not at the times it is most needed. The provision of power storage facilities (e.g., batteriès) will boost the capital cost of the wind power system.

Three different kinds of windmills can be considered for the aeration of ponds. These are Savonius type rotor, the American farm multivaned windmill, and the wind turbine (propeller) type windmill. The Darrieus vertical axis windmill cannot be used as it requires a high cut-in wind speed of more than 10 mph.

Wind power could be utilized in three different ways to aerate the ponds. They are:

(1) Agitating the liquid in the pond with a propeller immersed in the pond by providing a mechanical linkage between the propeller and the windmill rotor.

(2) Injecting compressed air into the pond from a air blower, which is driven by the windmill by a direct mechanical linkage between the blower and the windmill.

(3) Injecting air into the pond from a motor-driven blower or mechanical aerator using electricity generated by the windmill.

The feasibility of using the three different types of windmills is considered below.

The power potential of the different types of windmills was calculated at a design wind speed of 15 mph even though the mean monthly wind speed during July and August is only 9 mph. From Figure 15 it can be seen that the State of Iowa has wind speeds of 15 mph or more for about 4500 hours per year.

Savonius Type Rotor: Pondmaster Econo 672

As shown previously, the largest Pondmaster windmill, Econo 672, is capable of producing an output power of about 0.09 hp at a wind speed of 15 mph. If aeration is going to be needed it is essential to produce an output of about 1 hp. Moreover, this type of windmill has to be mounted directly over the pond. The vanes of this windmill are situated at modest height from ground level. If the pond is situated in an area surrounded by thick woods, not much power could be extracted at this low elevation. Therefore Pondmaster windmills are not feasible because of low power capacity and inflexible location.

American Farm Multivaned Windmills

These windmills have high starting torque and because of this they seem to be attractive if positive displacement type blowers are going to be used. Some manufacturers supply windmills with cut-in wind speeds as low as 5 mph (e.g. Dempster).

The Power potential of Dempster 14 ft windmill at 15 mph wind speed is shown as

$$\text{Theoretical Power} = \frac{\rho A V^3}{2} = \frac{1}{2} \times 2.25 \times 10^{-3} \frac{\text{lb sec}^2}{\text{ft}^4} \times \frac{\pi}{4} \times (14)^2 \text{ ft}^2 \times (15 \times \frac{22}{15})^3 \cdot \frac{\text{ft}^3}{\text{sec}^3}$$

$$= 1.84 \times 10^3 \frac{\text{lb}_f \text{ ft}}{\text{sec}}$$

$$= 3.35 \text{ hp}$$

If the efficiency is assumed to be 15%, actual power output = 0.15 × 3.35 = 0.5 hp. Therefore, to produce 1 hp, two windmills would be necessary.

Propeller Type Windmills

This type of windmill has comparatively higher efficiency and higher tip-to-wind-speed ratio. This type of windmill is feasible to use with centrifugal type blowers as they need high speed for starting. It is possible to find windmills of this type with cut-in speeds less than 7 mph (e.g. Aero Power Systems Model SL 1500, Energy Development Co. Model 440 & 445). But generally these types of windmills have design wind speeds around 25 mph.

To figure the diameter of the rotor required to produce 1 hp @ 15 mph wind speed, assume an efficiency of 30 percent and let D be the diameter of the rotor.

$$\text{Available power} = \frac{\rho A V^3}{2}$$

$$= \frac{1}{2} \times 2.25 \times 10^{-3} \frac{\pi}{4} \times D^2 \times (15 \times \frac{22}{15})^3 \frac{\text{lb}_f}{\text{sec}} \text{ ft}$$

$$= 9.4 D^2 \frac{\text{lb}_f \text{ ft}}{\text{sec}} = 1.71 \times 10^{-2} D^2 \text{ hp}$$

Output power $= 0.3 \times 1.71 \times 10^{-2} D^2$ hp

Needed power = 1 hp

$$D^2 = \frac{1}{0.3 \times 1.71 \times 10^{-2}} = 1.948 \times 10^2$$

$$D = 13.96 \text{ ft}$$

Required minimum diameter is 14 ft.

5. TYPICAL CAPITAL AND OPERATING COSTS

Even though the physical aspects do not appear favorable for wind power aeration of rest area stabilization ponds, a rough economic analysis will be presented here to illustrate the relative cost of wind power aeration versus purchased power aeration.

As shown in the preceding section, it will be cecessary to have a fairly large windmill to produce the 1 hp needed for aeration of the rest area ponds during the peak three-month summer season. For example, a wind turbine of 14 ft diameter was calculated for a design wind speed of 15 mph. To overcome the problem of low wind periods, a storage battery system will also be required. Equipment of this type is commercially available.

Approximate prices for such equipment were obtained from one company (Aero Power Systems, Inc., see Appendix). The current Model SL 1500 has a 12 ft diameter rotor which is different from that illustrated in the Appendix. The new Model SL 1500 has an output power of 158 kwhr per month at a mean wind speed of 10 miles per hour and 266 kwhr per month at a mean wind speed of 12 miles per hour. Thus, it is only marginally adequate and it might be necessary to operate the aeration equipment on a time clock during the night time and early morning hours when supplemental aeration would be most useful. Nevertheless, the cost of this unit will be used in the economic analysis. These costs are as follows (Aug. 1, 1979 price list):

SL 1500 Wind Turbine 12 volt DC	$2995
Tower, 50 ft high	700
Battery storage system, 4 battery set	660
Inverter 12 Volt DC to 120 volt AC	995
Shipping	100
Installation (estimate only)	1000
Total	$6450

The annual equivalent cost of this investment would be $753 per year. (n = 15 years, i = 8%, capital recovery factor = 0.1168). This annual cost must be compared with the cost of purchased power from a commercial power supplier.

Assume that a 1 hp floating aerator, such as that manufactured by Aqua Aerobics Systems, Inc., Rockford, Ill., was used in the pond, and assume that the aerator would be used only 6 months of the year during spring and summer and would operate 24 hours per day during those months. Also assume a motor efficiency of 90 percent. The power required from the power supplier would be

$$1 \text{ hp} \times 0.746 \frac{\text{kw}}{\text{hp}} \times 182 \text{ days} \times \frac{4 \text{ hr}}{\text{day}} \times \frac{1}{0.9 \text{ eff.}} = 3620 \text{ kwhr.}$$

The annual cost of the purchased power would depend on the price paid per kwhr. The costs at various typical power costs are as follows:

Power Rate $/kwhr	Annual Cost of power
0.04	$145
0.06	$217
0.08	$290
0.10	$362

Thus, the annual cost of purchased power is substantially less than the annual equivalent cost of wind power capital investment. If the aerators were operated less than 24 hours per day, the power costs would be proportionately less. Furthermore, the wind power cost does not include any allowance for maintenance of the wind power equipment.

The costs for the aeration equipment in the pond are assumed to be the same in both alternatives. The floating aerator was used in this comparison because such a device usually has lower initial cost and is slightly more efficient in oxygen transfer per unit power consumed than diffused aeration equipment. However, the power supply comparison would be equally valid for any type of aeration equipment selected.

Therefore, it must be concluded that the wind power alternative is not feasible on an economic basis. The use of wind power would reduce operating costs but the capital costs would be substantial and the annual equivalent cost of the capital would be far higher than the cost of purchased power at today's prices. The only way wind power could be selected would be in the situation where capital costs were of little concern compared to operating costs.

6. CONCLUSIONS

Evaluating the feasibility of using wind power to provide aeration of rest area stabilization ponds leads to the following conclusions:

1. Wind power theory presented herein can be used to estimate wind power development of different types of windmills with satisfactory accuracy.

2. The mean monthly wind speed in July and August in Iowa is 8 miles per hour (mph). Therefore, the cut-in wind speed of the windmill selected must be less than 8 mph. The design wind speed should be about 15 mph for maximum economy of power generation.

3. The wind patterns in Iowa are not ideal for the aeration power needs of the rest area ponds. In particular, the lowest monthly mean wind speeds occur in July and August, which are the months when traffic load is heaviest on the interstate highways. Also, the wind speeds are lowest in night time and early morning and highest in the afternoon, which is the reverse of what is desired for aeration.

4. Because of the unfavorable wind patterns, a power storage system would be required to provide the power during those hours when wind speeds are not sufficient to meet the aeration power needs in an optimal fashion.

5. The small inexpensive floating wind-driven aeration devices which stimulated the interest in this project cannot generate sufficient power to meet the aeration power requirements for the rest area ponds.

6. To provide the aeration power required for the ponds (about 1 hp/0.25 acre cell), a tower-mounted wind turbine with a diameter of 14

would be required at 15 mph wind speed. Alternatively, two 14 ft diameter multivane American farm type windmills would be required.

7. The annual equivalent cost for a 14 ft diameter wind turbine with storage battery system and inverter to convert DC to AC power is estimated to be $753 per year (n=15 yrs, i=0.08%). Purchased power costs range from $145 per year at 4 cents/kwhr to $362 per year at 10 cents/kwhr assuming operation of a 1 hp floating aerator 182 days per year, 24 hrs per day.

8. Therefore, wind powered aeration of the rest area stabilization ponds is feasible but is not economically justified at today's power costs, unless capital costs are ignored.

7. REFERENCES

1. Babcock, W.H., A.L. Frey and W.L. Somerell. Wind Powered Aeration for Remote locations. Workshop on Wind Energy Conversion Systems. Washington, D.C. June 9-11, 1975.

2. Galanis, N., S. Nasariah and C.C. Dang. Use of Wind Energy for the aeration of Waste Waters: A Case Study. U.S. National Conference on Wind Engineering Research. Fort Collins, Colorado. June 22-25, 1975.

3. Golding, E.W. The Generation of Electricity by Wind Power. New York: Philosophical Library, 1956.

4. Hutter, U. Wind Power Machines. Microfilm No. n-75-17786, NASA-TT-F-16195, NASA, Washington, D.C., Feb. 1975.

5. Loftness, R.L. Energy Handbook. New York: Van Nostrand Reinhold Company, 1975.

6. Penner, S.S. and L. Icerman. Energy Volume II Non-Nuclear Energy Technologies. New York: Addison-Wesley Publishing House, 1975.

7. Savonius, S.J. The S-Rotor and its Applications. Mechanical Engineering, 53(5), 333-338, 1931.

8. The United Nations. New Source of Energy and Economic Development. Department of Economic and Social Affairs. The United Nations, New York, New York, 1957.

9. The United States Department of Commerce, Weather Bureau. Local Climatological Data, Des Moines, Iowa, 1955, 1964.

10. Wegley, H.L., M.M. Orgill and R.L. Drake. A Siting Handbook for Small Wind Energy Conversion Systems. Pacific Northwest Laboratory, Richland, Washington 99352, May 1978.

11. White, P.J. Iowa Wind Pattern. Iowa Geographer, Nov. 30, 1972, 29-36.

8. APPENDIX. DESCRIPTIVE INFORMATION ABOUT COMMERCIALLY AVAILABLE WIND MACHINES

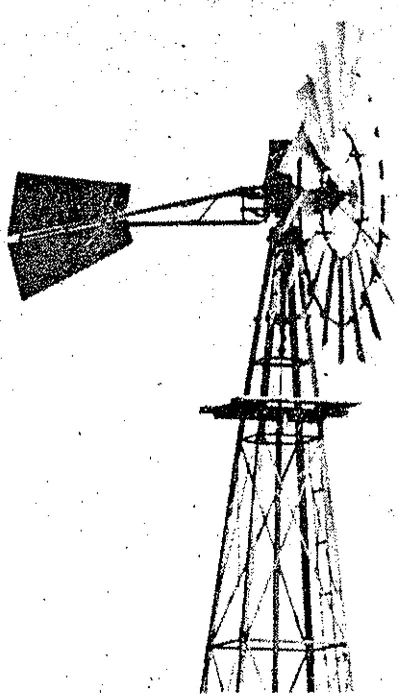
Model: 702-14 ft.

Rotor Diameter: 14 feet
Rotor Weight: Not available
System Weight: 1695 lbs.
Blade Materials: Galvanized steel
Cut-in Wind Speed: 9 Mph.
Shut-down Wind Speed: 28 Mph.
Rated Output: See chart, 15-20 Mph.
Maximum Output: See chart, 15-20 Mph.
RPM at Rated Output: 62
Overspeed control: Rotor turns sideways to the wind
Testing Procedures: 44 years of manufacturing
Warranty: One year, materials and workmanship
Maintenance Schedule: Annual lubrication

Model: 702-16 ft.

Rotor Diameter: 16 feet
Rotor Weight: Not available
System Weight: 2450
Blade Materials: Galvanized steel
Cut-in Wind Speed: 9 Mph.
Shut-down Wind Speed: 28 Mph.
Rated Output: See chart, 15-20 Mph.
Maximum Output: See chart, 15-20 Mph.
RPM at Rated Output: 53
Overspeed control: Rotor turns sideways to the wind
Testing Procedures: 44 years of manufacturing
Warranty: One year, materials and workmanship
Maintenance Schedule: Annual lubrication

Dempster



Manufacturer: Dempster Industries, Inc.

Address: P.O. Box 848, Beatrice, NB 68310

Contact: Sales Department

Telephone: 402-223-4026

Machine Description: Up-wind, horizontal-axis, water-pumpers.

Model: 6 ft.

Rotor Diameter: 6 feet
Rotor Weight: 100 lbs.
System Weight: 280 lbs.
Blade Materials: Galvanized steel
Cut-in Wind Speed: 5 Mph.
Shut-down Wind Speed: 50 Mph.
Rated Output: See chart, 15 Mph.
RPM at Rated Output: Not available
Overspeed control: Rotor turns sideways to the wind
Testing Procedures: Data calculated and tested
Warranty: Limited five years, parts and workmanship
Maintenance Schedule: Annual inspection and lubrication

Model: 8 ft.

Rotor Diameter: 8 feet
Rotor Weight: 120 lbs.
System Weight: 388 lbs.
Blade Materials: Galvanized steel
Cut-in Wind Speed: 5 Mph.
Shut-down Wind Speed: 50 Mph.
Rated Output: See chart, 15 Mph.
RPM at Rated Output: Not available
Overspeed control: Rotor turns sideways to wind
Testing Procedures: Calculated and tested
Warranty: Limited five years, parts and workmanship
Maintenance Schedule: Annual inspection and lubrication

Model: 10 ft.

Rotor Diameter: 10 feet
Rotor Weight: 150 lbs.
System Weight: 500 lbs.
Blade Materials: Galvanized steel
Cut-in Wind Speed: 5 Mph.
Shut-down Wind Speed: 50 Mph.
Rated Output: See chart, 15 Mph.
RPM at Rated Output: Not available
Overspeed control: Rotor turns sideways to wind
Testing Procedures: Data calculated and tested
Warranty: Limited five years, parts and workmanship
Maintenance Schedule: Annual inspection and lubrication

Model: 12 ft.

Rotor Diameter: 12 feet
Rotor Weight: 334 lbs.
System Weight: 935 lbs.
Blade Materials: Galvanized steel
Cut-in Wind Speed: 5 Mph.
Shut-down Wind Speed: 50 Mph.
Rated Output: See chart, 15 Mph.
RPM at Rated Output: Not available
Overspeed control: Rotor turns sideways to the wind
Testing Procedures: Data calculated and tested
Warranty: Limited five years, parts and workmanship
Maintenance Schedule: Annual inspection and lubrication

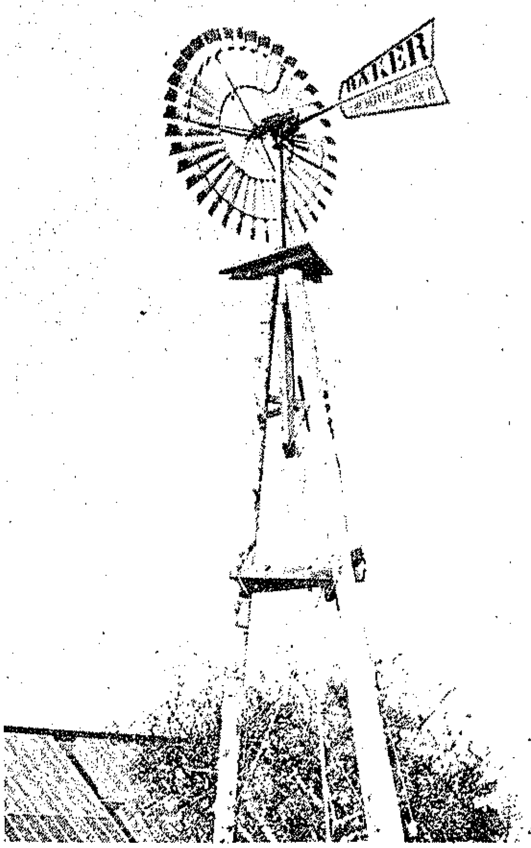
Model: 14 ft.

Rotor Diameter: 14 feet
Rotor Weight: 616 lbs.
System Weight: 1450 lbs.
Blade Materials: Galvanized steel
Cut-in Wind Speed: 5 Mph.
Shut-down Wind Speed: 50 Mph.
Rated Output: See chart, 15 Mph.
RPM at Rated Output: Not available
Overspeed control: Rotor turns sideways to the wind
Testing Procedures: Data calculated and tested
Warranty: Limited 5 years, parts and workmanship
Maintenance Schedule: Annual inspection and lubrication

DEMPSTER PUMPING CAPACITIES
(15 Mile-Per-Hour Wind)

Cylinder	6 Ft. 5" Stroke		8 Ft. "A" 7 ½" Stroke		10 Ft. 7 ½" Stroke		12 Ft. 12" Stroke		14 Ft. 12" Stroke	
Size	Elev.	G.P.H.	Elev.	G.P.H.	Elev.	G.P.H.	Elev.	G.P.H.	Elev.	G.P.H.
1 7/8"	120	115	172	173	256	140	388	180	580	159
2"	95	130	135	195	210	159	304	206	455	176
2 1/4"	75	165	107	248	165	202	240	260	360	222
2 1/2"	62	206	89	304	137	248	200	322	300	276
2 3/4"	54	248	77	370	119	300	173	390	260	334
3"	45	294	65	440	102	357	147	463	220	396
3 1/4"	39	346	55	565	86	418	125	544	187	465
3 1/2"	34	400	48	600	75	487	108	630	162	540
3 3/4"	29	457	42	688	65	558	94	724	142	620
4"	26	522	37	780	57	635	83	822	124	706

Heller-Aller



Manufacturer: Heller-Aller Company

Address: Perry and Oakwood Streets, Napoleon, Ohio 43545

Contact: James Bradner, vice-president

Machine Description: Up-wind, horizontal-axis, water-pumpers.

Model: Baker 6 ft.

Rotor Diameter: 6 feet
Rotor Weight: Not available
System Weight: 220 lbs.
Blade Materials: Galvanized steel
Cut-in Wind Speed: 7 Mph.
Cut-out Wind Speed: 25 Mph.
Rated Output: See chart, 15 Mph.
RPM at Rated Output: 150
Overspeed control: Rotor turns sideways to the wind
Testing Procedures: Not available
Warranty: One year, parts and workmanship
Maintenance Schedule: Annual inspection and lubrication

Model: Baker 8 ft.

Rotor Diameter: 8 feet
Rotor Weight: Not available
System Weight: 360 lbs.
Blade Materials: Galvanized steel
Cut-in Wind Speed: 7 Mph.
Cut-out Wind Speed: 25 Mph.
RPM at Rated Output: 150
Overspeed control: Rotor turns sideways to wind
Testing Procedures: Not available
Warranty: One year, parts and workmanship
Maintenance Schedule: Annual inspection and lubrication

Model: Baker 10 ft.

Rotor Diameter: 10 feet
Rotor Weight: Not available
System Weight: 475 lbs.
Blade Materials: Galvanized steel
Cut-in Wind Speed: 7 Mph.
Shut-down Wind Speed: 25 Mph.
Rated Output: See chart, 15 Mph.
RPM at Rated Output: 150
Overspeed control: Rotor turns sideways to the wind
Testing Procedures: Not available
Warranty: One year, parts and workmanship
Maintenance Schedule: Annual inspection and lubrication

Model: Baker 12 ft.

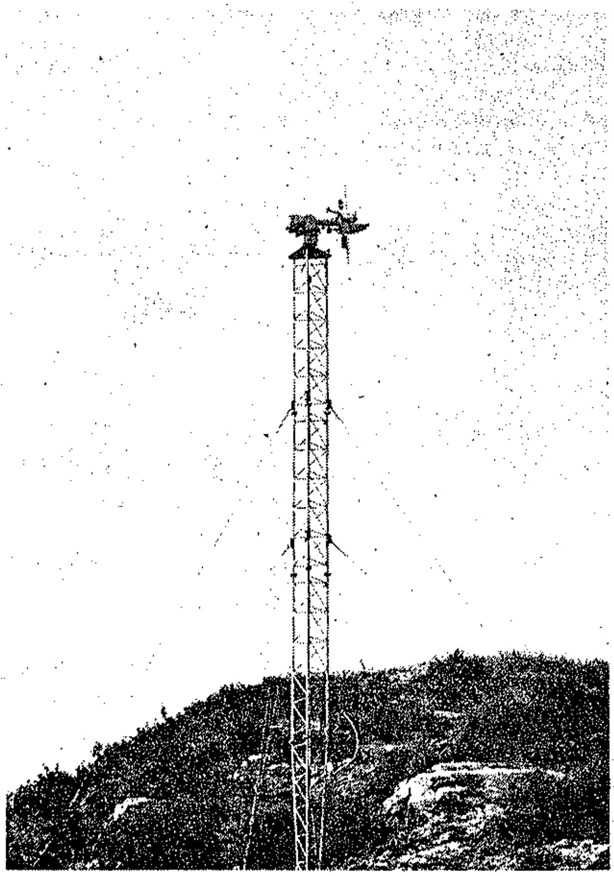
Rotor Diameter: 12 feet
Rotor Weight: Not available
System Weight: 800 lbs.
Blade Materials: Galvanized steel
Cut-in Wind Speed: 7 Mph.
Shut-down Wind Speed: 25 Mph.
Rated Output: See chart, 15 Mph.
RPM at Rated Output: 150
Overspeed control: Rotor turns sideways to the wind
Testing Procedures: Not available
Warranty: One year, parts and workmanship
Maintenance Schedule: Annual inspection and lubrication

	PUMPING CAPACITIES (In A 15-Mile-Per-Hour Wind)							
	6 Foot Baker		8 Foot Baker		10 Foot Baker		12 Foot Baker	
Total Elevation In Feet	Diameter of Cylinder (Inches)	U.S. Gallons Per Hour	Diameter of Cylinder (Inches)	U.S. Gallons Per Hour	Diameter of Cylinder (Inches)	U.S. Gallons Per Hour	Diameter of Cylinder (Inches)	U.S. Gallons Per Hour
25	3	350	3 1/2	900	4	1250	6	2400
35	2 1/2	240	3	720	3 1/2	925	5	1625
50	2 1/4	200	2 1/2	450	3	700	4 1/2	1425
75	2	160	2 1/4	350	2 1/2	475	4	1125
100	2	150	2	250	2 1/2	460	3	600
125	1 5/8	120	1 7/8	240	2	280	2 1/2	525
150			1 3/4	220	2	280	2 1/2	525
200					1 7/8	260	2	325
250					1 3/4	215	2	325
300							1 3/4	200

The above capacities are approximate. By the total elevation in feet, we do not mean the depth of the well, but the distance to the cylinder. Do not use pipe smaller than that for which cylinders are fitted.

While we recommend the above table, larger cylinders may in many circumstances, be used with satisfaction.

Storm Master



Manufacturer: Wind Power Systems, Inc.

Address: P.O. Box 17323, San Olego, CA 92117

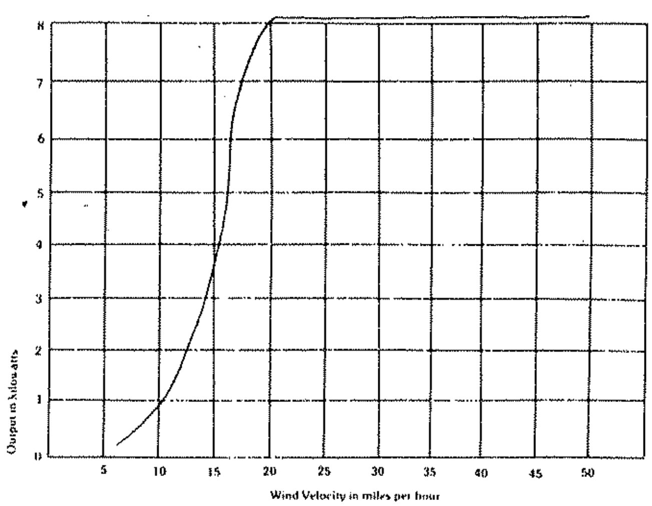
Contact: Ed Salter

Telephone: 714-452-7040

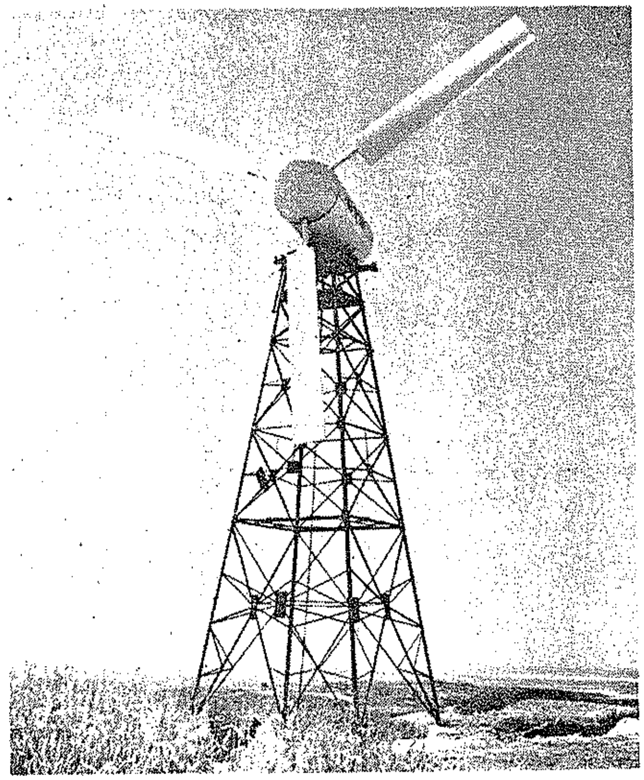
Machine Description: Down-wind; horizontal-axis, three blades.

Storm Master 10

Rotor Diameter: 32.8 feet
Rotor Weight: 285 lbs.
System Weight: 875 lbs.
Blade Materials: Fiberglass shell, foam core
Cut-in Wind Speed: 8 Mph.
Shut-down Wind Speed: 150 Mph.
Rated Output: 6000 watts at 18 Mph.
Maximum Output: 6000 watts
RPM at Rated Output: 130
Overspeed control: Blade stall, brake
Generator Alternator: Variety available, including permanent magnet
Testing Procedures: Data calculated
Warranty: One year, materials and workmanship
Maintenance Schedule: Not available



WTG Systems

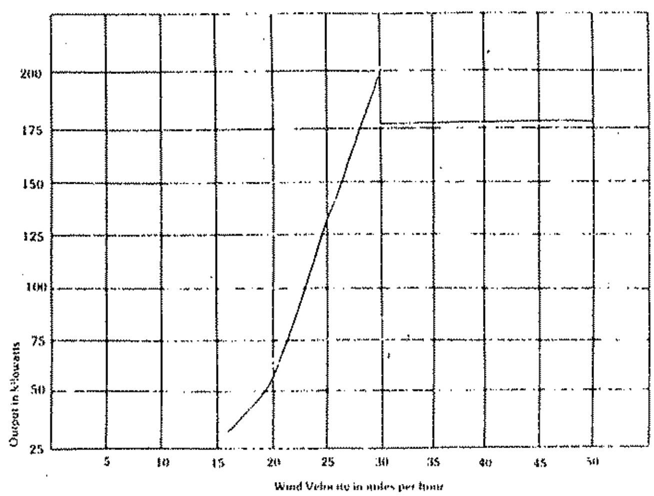


Manufacturer: WTG Energy Systems
Address: Box 87, 1 LaSalle St. Angola, N.Y. 14006
Contact: Alfred J. Gross — Director of Marketing
Telephone: 716-549-5544

Machine Description: Up-wind, horizontal-axis, three blades.

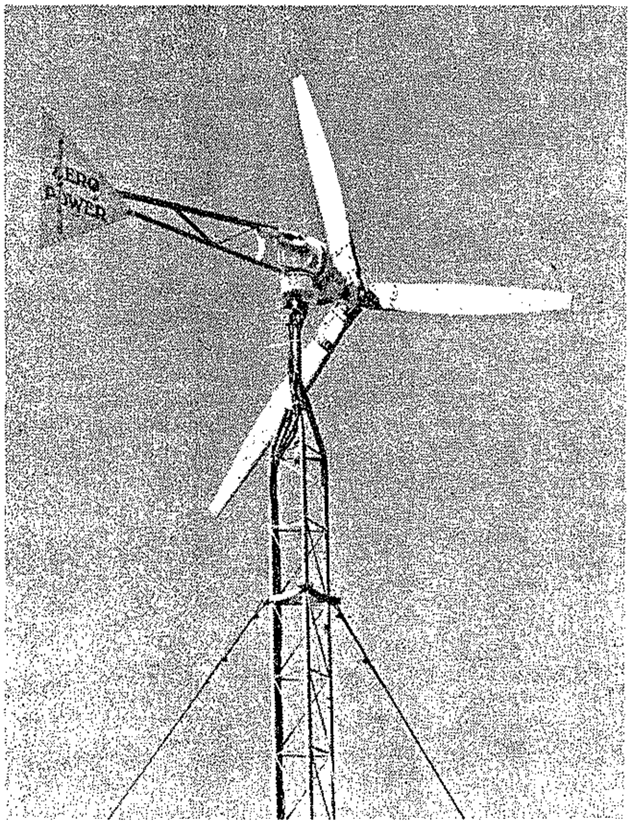
Model: MP 1-200

Rotor Diameter: 80 feet
Rotor Weight: 15,000 lbs.
System Weight: 85,000 lbs.
Blade Materials: Steel, steel tubing, galvanized steel
Cut-in Wind Speed: 8 Mph.
Cut-out Wind Speed: 50 Mph.
Rated Output: 200,000 watts
Maximum Output: 200,000 watts
Generator / Alternator: Synchronous generator
Testing Procedures: During operation



Aero Power Systems

Model: SL 1500



Manufacturer: Aero Power Systems, Inc.

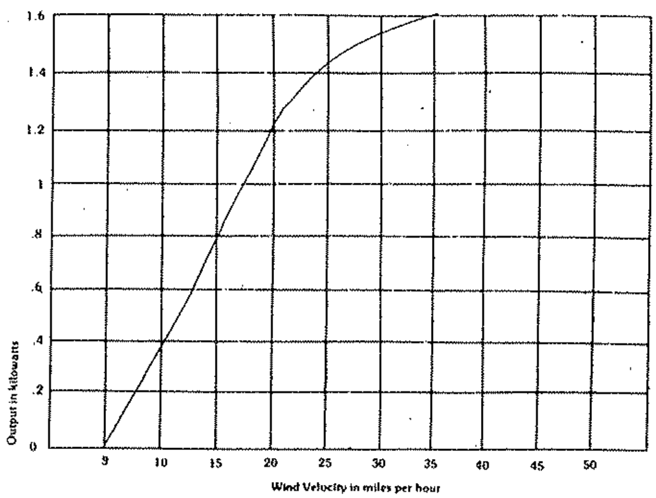
Address: 2398 Fourth Street, Berkeley, CA 97410

Contact: Mario Agnello

Telephone: 415-848-2710

Machine Description: Up-wind, horizontal-axis, three blades.

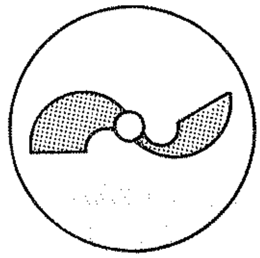
Model: SL 1500
Rotor Diameter: 10 feet
Rotor Weight: 50 lbs.
System Weight: 160 lbs.
Blade Materials: Wood, sitka spruce
Cut-in Wind Speed: 6 Mph.
Shut-down Wind Speed: 100 Mph.
Rated Output: 1430 watts at 25 Mph.
Maximum Output: 1600 watts at 30 Mph.
RPM at Rated Output: 500
Overspeed control: Mechanical, variable pitch, centrifugally activated.
Generator / Alternator: 14 or 28 VAC, 3 phase, DC output
Testing Procedures: Field operation
Warranty: One year, defects in workmanship and materials.
Maintenance Schedule: Semi-annual, grease hub, check blades



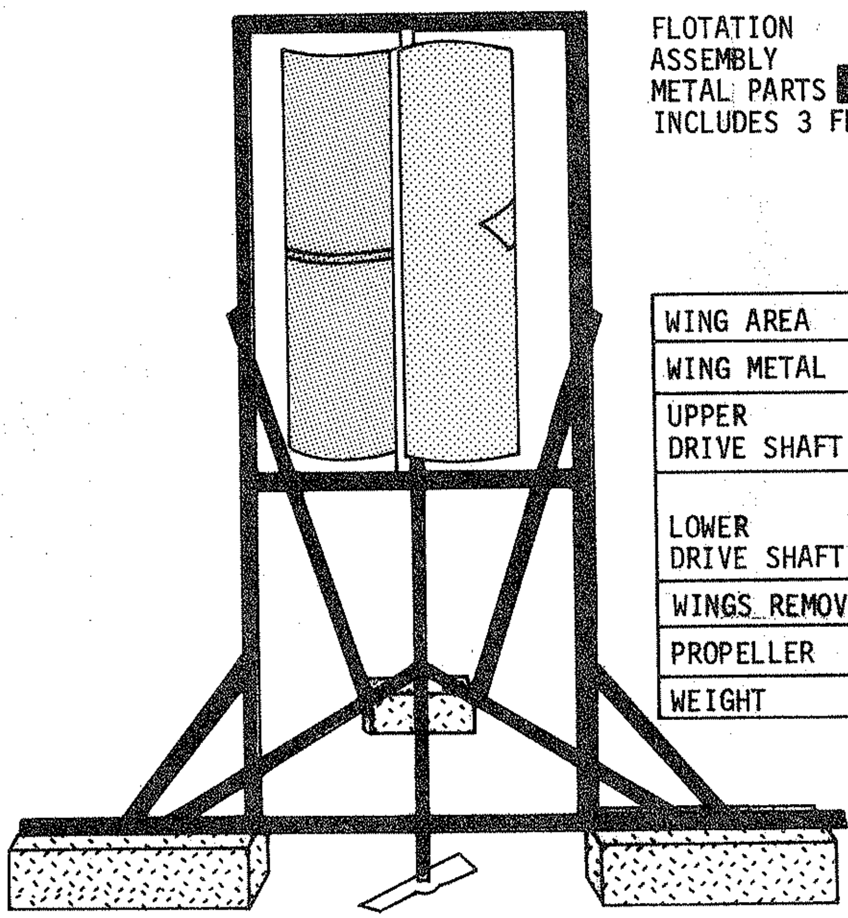
PONDMASTER 271

TOP VIEW OF THE MODEL 271 SHOWING THE 2 WIND DRIVEN WINGS, 3 1/2 ft. TALL.

MODEL 271
TOP VIEW



ECONO
271



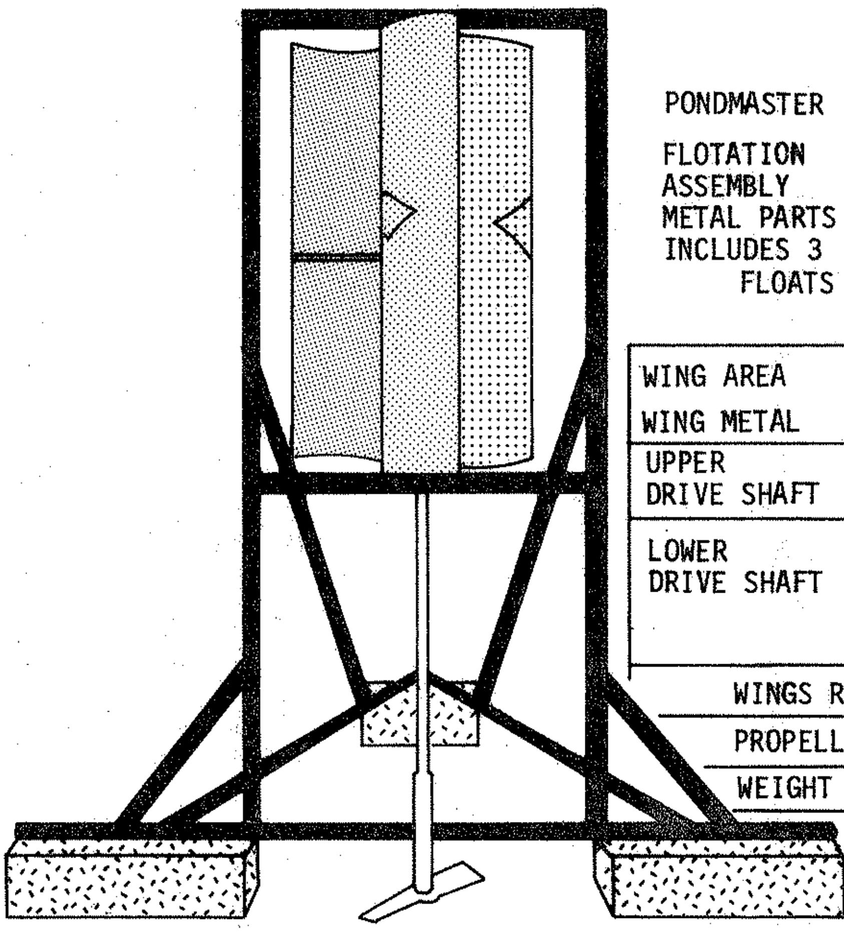
PONDMASTER	MODEL	WT.	LIST PRICE
	271	16 lbs.	$39.95
FLOTATION ASSEMBLY METAL PARTS INCLUDES 3 FLOATS		51 lbs.	39.95
		TOTAL	$79.90

WING AREA	10 1/2 Sq. ft.
WING METAL	ALUMINUM
UPPER DRIVE SHAFT	1/2" STEEL
LOWER DRIVE SHAFT	SINGLE PROTECTED AGAINST FREEZING OIL PRESSURE ONLY
WINGS REMOVABLE	YES
PROPELLER	2 BLADE 18"
WEIGHT	20 lbs.

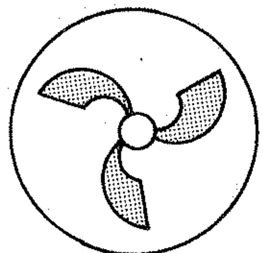
PONDMASTER 370

TOP VIEW SHOWING THE 3 WINGS ON THE MODEL 370.
WINGS ARE 3 1/2 ft TALL.

STANDARD
370



MODEL 370
TOP VIEW

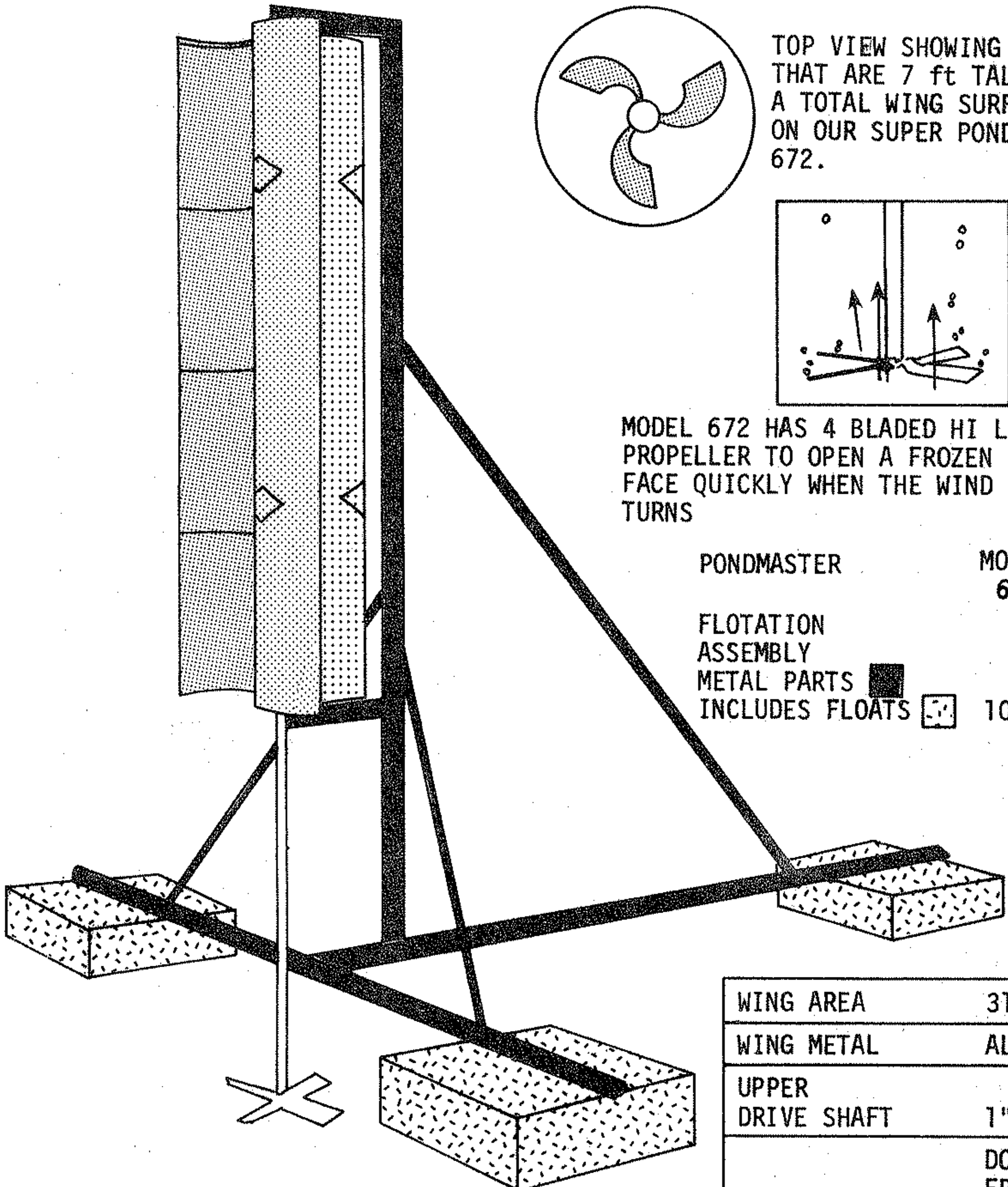


PONDMASTER	MODEL	WT.	LIST PRICE
	370	26 lbs.	$74.95
FLOTATION ASSEMBLY METAL PARTS ■ INCLUDES 3 FLOATS ⬚	771	51 lbs	39.95
		TOTAL	$114.90

WING AREA	15 3/4 Sq. ft.
WING METAL	ALUMINUM
UPPER DRIVE SHAFT	1" x .250 WALL ALUMINUM
LOWER DRIVE SHAFT	DOUBLE-LOWER SHAFT FREE WHEELING, NEOPRENE SEAL PLUS OIL PRESSURE SEALED
WINGS REMOVABLE	YES
PROPELLER	2 BLADE 18"
WEIGHT	26 lbs.

PONDMASTER 672

SUPER
672



TOP VIEW SHOWING THE 3 WINGS THAT ARE 7 ft TALL WHICH GIVES A TOTAL WING SURFACE OF 31 ft ON OUR SUPER PONDMASTER MODEL 672.

MODEL 672 HAS 4 BLADED HI LIFT PROPELLER TO OPEN A FROZEN SURFACE QUICKLY WHEN THE WIND RETURNS

	MODEL	WT.	LIST PRICE
PONDMASTER	672	40 lbs.	$109.95
FLOTATION ASSEMBLY METAL PARTS INCLUDES FLOATS	1072	80 lbs.	79.95
		TOTAL	$189.90

WING AREA	31 Sq. ft.
WING METAL	ALUMINUM
UPPER DRIVE SHAFT	1" STEEL
LOWER DRIVE SHAFT	DOUBLE-LOWER SHAFT FREE WHEELING, NEOPRENE SEAL PLUS OIL PRESSURE SEALED
WINGS REMOVABLE	YES
WEIGHT	40 lbs.
PROPELLER	4 BLADE 18"

IOWA STATE UNIVERSITY

Department of Civil Engineering
Ames, Iowa 50011

Telephone: 515-294-3532

Mr. Harold Dolling
Highway Division
Iowa Dept. of Transportation
800 Lincoln Way
Ames, Iowa 50010

RE: Travel trailer disposal load to Interstate Rest Area lagoons.

Dear Harold:

I have the information you provided on the counts of trailers dumping at the disposal stations and the interview responses. (Your letter of 9/6/79 and attached data sheets). I also have the BOD test results for the composite samples collected during the survey. It will not be possible to include an analysis of the above information in our final report on the HR-207 contract due to the deadline for submittal. However, I'll attempt an analysis here and submit it for your information and use along with the final report.

The data presented confirms my expectation of wide variability in the frequency of usage, the volume dumped per trailer and the BOD concentration. So it is still difficult to propose a realistic load from these disposal stations. Nevertheless, I offer the following approach:

Frequency of use:	Range	Average	Standard Deviation	Number of Data Points
Week day data				
Uses per day per station	3-10	6	3.0	6
Weekend data				
Uses per day per station	32-39	35	--	2

Therefore, since the weekend data was collected on Sunday near a major city, assume it is representative only of the Sunday load, and the weekday data are typical of the other six days. The weighted average frequency of use would then be:

$$(6 \text{ days} \times 6 + 1 \text{ day} \times 35)/ 7 = 10 \text{ uses/day/station.}$$

Volume discharged per use

	Range	Average	Standard Deviation	Number of Data Points
Gallons per discharge	5-40	19.5	11.8	17

BOD Concentration of Discharges

BOD_5 mg/l	390-7500	3270	2600	12

Now, if we use the average values of these items in calculating the load, we obtain:

$$10 \frac{\text{uses}}{\text{day}} \times 19.5 \frac{\text{gallons}}{\text{use}} \times 3.78 \frac{1}{\text{gal}} \times 3.270 \frac{g}{l} = 2410 \text{ g } BOD_5/\text{day}$$

$$= 5.31 \text{ lb } BOD_5/\text{day}.$$

Surprisingly, this is quite close to the 5 lb BOD_5/day estimated previously in the final report. However, the data collected give us a better feeling for the potential range of the load. For example, if a particular station receives both high week day usage and high weekend usage, and if the upper values of volume dumped and BOD concentration prevailed at that station, the load could be as high as:

$$\left(\frac{10 \times 6 + 35 \times 1}{7} \right) \frac{\text{uses}}{\text{day}} \times 40 \frac{\text{gal}}{\text{use}} \times 3.87 \frac{1}{\text{gal}} \times 7.5 \frac{g}{l} = 15390 \frac{g\ BOD_5}{\text{day}}$$

$$= 33.9 \frac{\text{lb } BOD_5}{\text{day}}$$

While this combination of events is not very likely, it is a remote possibility.

Therefore, I suggest we assume the 5 lb BOD_5/ day in the final report is still reasonable. The "wait and see" approach outlined in the report will permit you to identify the ponds creating nuisance conditions. Those conditions may be due to high travel trailer disposal load. When such conditions are observed, aeration equipment or larger ponds will be required.

In retrospect, I wish we had asked for COD (Chemical Oxygen Demand) tests as well as BOD tests on the samples. That might have given us a better idea of the impact of the chemicals used in the toilets on the BOD results. If you do any more sampling, run both the COD and BOD on any future samples.

Sincerely,

John L. Cleasby

John L. Cleasby
Professor of Civil Engineering